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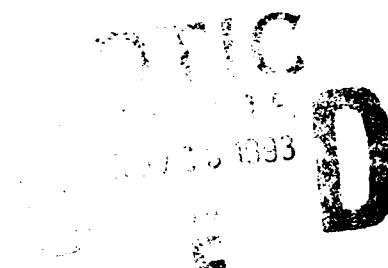
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Repair, Evaluation, Maintenance, and Rehabilitation Research Program

Comparison of Cast-in-Place Concrete Versus Precast Concrete Stay-in-Place Forming Systems for Lock Wall Rehabilitation

by *William R. Miles*
Donald J. Bergmann & Associates, P.C.



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	<u>Problem Area</u>		<u>Problem Area</u>
CS	Concrete and Steel Structures	EM	Electrical and Mechanical
GT	Geotechnical	EI	Environmental Impacts
HY	Hydraulics	OM	Operations Management
CO	Coastal		

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by William R. Miles

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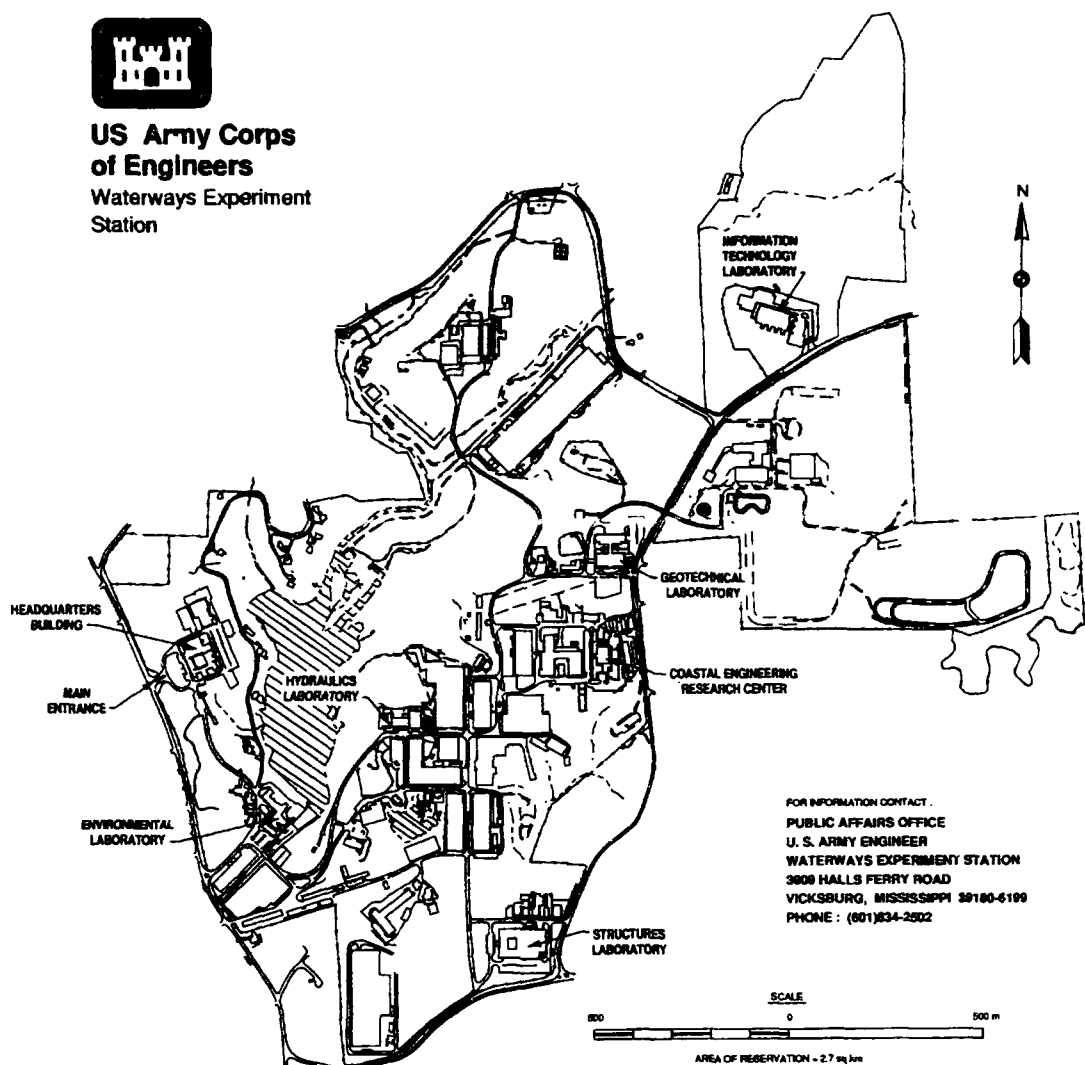
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Preface

The study reported herein was authorized by Headquarters, U.S. Army Corps of Engineers (HQUSACE), under Civil Works Research Work Unit 32636, "New Concepts in Maintenance and Repair of Concrete Structures," for which Mr. James E. McDonald, U.S. Army Engineer Waterways Experiment Station (WES), Structures Laboratory (SL), is Principal Investigator. This work unit is part of the Concrete and Steel Structures Problems Area of the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program sponsored by HQUSACE, for which Mr. McDonald is the Problem Area Leader. The Overview Committee at HQUSACE for the REMR Research Program consists of Mr. James E. Crews (CECW-O) and Dr. Tony C. Liu (CECW-EG). Technical Monitor for this study was Dr. Liu. Mr. William N. Rushing (CERD-C) was the REMR Coordinator at the Directorate of Research and Development, HQUSACE. Program Manager for REMR is Mr. William F. McCleese (CEWES-SC-A).

The study was performed by Bergmann Associates, Rochester, NY, under contract to the U.S. Army Engineer District, New York (USAEDNY). The work was conducted under the general supervision at WES of Mr. Bryant Mather, Director, SL, and Mr. Kenneth L. Saucier, Chief, Concrete Technology Division (CTD), and under the direct supervision of Mr. McDonald and Mr. Bill Petronis, Albany Field Office, USAEDNY. This report was prepared by Mr. William R. Miles with review by Messrs. Robert J. Radley and Kenneth P. Allen, all of Bergmann Associates. The author would like to thank Messrs. McDonald and Petronis, Messrs. Rich Campbell and Al Ellinwood, Albany Field Office, and Messrs. H. John Marcelle and Peter J. Smith, Fort Miller Precast Company, Schuylerville, NY, for their cooperation and assistance.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Bruce K. Howard, EN.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
degrees (angle)	0.01745329	radians
Fahrenheit degrees	5/9	Celsius degrees or kelvins ¹
feet	0.3048	metres
gallons (US liquid)	3.785412	litres
inches	25.4	millimetres
kips (force) per square inch	6.894757	megapascals
miles (US statute)	1.609344	kilometres
pounds	0.4535924	kilograms
pounds (force) per square inch	0.006894757	megapascals
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
¹ To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: $C = (5/9)(F - 32)$. To obtain Kelvin readings, use $K = (5/9)(F - 32) + 273.15$.		

1 Introduction

Project Description

The Troy Lock and Dam is located on the Hudson River in the City of Troy, Rensselaer County, NY. This facility is the only lock and dam included in the Federal improvements on the Hudson River. The 14-ft¹ deep channel is maintained from the Port of Albany to the south to the Village of Waterford, NY, to the north. The confluence of the Mohawk and Hudson Rivers is located at Waterford, as is the east end of the New York State Barge Canal and the south end of the Champlain Canal.

The current facilities were originally constructed in 1915 and opened for navigation in the Spring of 1916, replacing a State of New York facility located 1,400 ft downstream. After approximately 60 years in service, the condition of the lock and dam was evaluated by the U.S. Army Engineer Waterways Experiment Station (USAEWES). Rehabilitation of the lock and dam was recommended based on the results of an engineering condition survey (Pace 1978) and a structural evaluation (Pace, Campbell, and Wong 1981). Later inspection work by Bergmann Associates, Rochester, NY, in 1984, verified the need for remedial actions.

An interim repair program was designed by Bergmann Associates in 1987, and construction was completed in 1990. Repairs were made to the lock wall monoliths adjacent to the miter gates, lock operating house, culverts, valves, and headgate area concrete.

The second phase of the repair program, also designed by Bergmann Associates, was constructed by Jackie Bombard, Inc., general contractor, Galway, NY, and completed in the Fall of 1992. The scope of work in this phase includes:

¹ A table of factors for converting non-SI units of measurements to SI (metric) units is presented on page viii.

- a. Concrete repairs to monoliths 4 through 22 and 35 through 53, including aprons and lock chamber wall above elevation (el) -2.¹
- b. Concrete repairs to monoliths 39 through 55, including aprons and river face of the wall above el -2 downstream of the dam.
- c. Concrete repairs to monoliths 32 through 39, including aprons and river face of the wall above el 12 upstream of the dam.
- d. New line poles and ladders in the lock chamber and resetting of existing snubbing posts, railings, and pins. New railings were installed in some areas.

All repair work on this project has been performed on the 44.4-ft - wide by 492.5-ft - long by 14-ft - draft lock chamber which is located on the east bank of the Hudson River.

Concrete repairs in the interim repair program were performed using cast-in-place concrete techniques, while the repairs in the second phase used a combination of cast-in-place and precast concrete stay-in-place forming systems. Precast concrete was utilized on the lock chamber walls and on the river wall. Panel details are included as Appendix A of this report, and panel contract specifications are included as Appendix B.

Objectives

The objectives of this report are as follows:

- a. To present the design and details utilized on the precast concrete panel rehabilitation at Troy Lock.
- b. To compare the results of precast concrete panel rehabilitation with previous cast-in-place repairs, with respect to quality, cost, and schedule.
- c. To recommend refinements in the details and methods of fabrication and erection of the precast panels, based on lessons learned from work performed at Troy Lock.

¹ All elevations (el) cited herein are in feet referred to the National Geodetic Vertical Datum (NGVD) of 1929.

2 Background Information

Previous Studies

The general approach in lock wall rehabilitation has been to remove 1 to 3 ft of concrete from the face of the lock wall and to replace it with air-entrained concrete by using conventional forming and placing procedures. While conventional cast-in-place concrete repairs have several advantages, one of the most persistent problems in lock wall rehabilitation with this approach is cracking in the replacement concrete (McDonald 1987). These cracks, which generally extend completely through the conventional replacement concrete, are attributed primarily to restraint of volume changes resulting from shrinkage, thermal gradients, and autogenous volume changes. In most cases, such cracking will not cause structural deficiencies; however, the cracks are unsightly and often require additional maintenance to minimize deterioration of reinforcement.

One approach to minimizing the cracking problem is to use precast concrete panels as stay-in-place forms. A precast panel rehabilitation system was designed by ABAM Engineers, Federal Way, WA, in Phase I of a contract with the USAEWES (ABAM 1987a). Phase II was a constructability demonstration in which eight panels were precast and erected on two simulated lock wall monoliths at WES (ABAM 1987b). With few exceptions, the design criteria and details developed in this work were incorporated into the design of the lock wall resurfacing system for Lock 22, Mississippi River.

As part of the 30 percent Design Submission for the Troy Lock Concrete Repair Project, Bergmann Associates, in a joint effort with the US Army Engineer District, New York (USAEDNY), Albany Field Office (AFO), evaluated conventional cast-in-place concrete and precast concrete stay-in-place forms for lock wall repairs. The purpose of this report (Radley 1990) was to compare the two methods and to recommend construction details to be incorporated in the Concrete Repair Project based on initial cost, constructability, and durability.

The 1990 report recommended the use of precast panels as well as modifications to the original design used at Lock 22. Reasons for the recommendation included:



Figure 1. Troy Lock prior to rehabilitation



Figure 2. Troy Lock walls prior to rehabilitation

- a. Cost estimates showed the total cost for each system to be approximately equal.
- b. Construction was judged to be easier for the precast panels due to the elimination of formwork, elaborate heated enclosures, reduced placement of bar reinforcement in the field, and less field quality control work.
- c. Precast panels produced in the controlled environment of a precast plant would have higher quality concrete, with increased compressive strength, abrasion resistance, and impermeability.
- d. Precast panels would be more durable and would be expected to outlive cast-in-place construction due to the impervious, crack-free surface provided which minimizes the chance of water penetration and subsequent deterioration of embedded reinforcing steel.
- e. The higher compressive strength and abrasion resistance typical of the precast panel will provide a more attractive lock wall surface.

Concrete Repair at Mississippi River Lock 22, Hannibal, MO

The US Army Engineer District, Rock Island, completed a lock chamber rehabilitation project in the Winter of 1988-1989 using precast panel technology at Lock 22 on the upper Mississippi River, near Hannibal, MO. The lock chamber at Lock 22 is 110 ft wide by 600 ft long with a 10 -ft, 6 -in. lift. The facility was initially constructed in 1935, and the rehabilitation project included \$10,000,000 in repairs to concrete work and mechanical/electrical systems.

Rehabilitation included a single row of panels at a 10 -ft, 6 -in. height with maximum lengths in the 30 -ft range. Panels were 6-1/2 in. in thickness with a minimum infill concrete thickness of 5-1/2 in. Panels were held in place through means of a welded connection between dowel anchors and a series of embedded plates located on the top and bottom faces. A jack screw system below each panel was used for leveling of the panels into final position.

A visit was made to the Lock 22 construction site in January 1989 by Rich Campbell and Al Ellinwood (USAEDNY, AFO) and Ken Allen (Bergmann Associates). The purpose of that visit was to gain first-hand observations of the precast panel technology and its applicability to future construction at the Troy Lock and Dam.

Concrete Repair at Troy Lock, Troy, NY

Conditions at Troy Lock prior to rehabilitation are shown on Figures 1 and 2.

Phase I work (Interim Repair Program) at Troy Lock for the USAEDNY was completed in 1990 and included the following items of work:

- a.* Cast-in-place concrete repairs to monoliths 1, 2, 3, 23, 24, 32, 33, 34, 54, and 55. These are the monoliths adjacent to the upstream and downstream miter gates.
- b.* Concrete repair to the south and west elevations of the lock operating house.
- c.* Culvert and culvert port repairs.
- d.* Valve rehabilitation.
- e.* Miscellaneous surface repairs.
- f.* Scour holes filled at the dam.
- g.* Concrete repairs to the headgates on the west side of the river.
- h.* Dewatering and repairs to the normal upstream winter needle dam area through cellular cofferdam.

Phase II Work (Concrete Repair Project) at Troy Lock was completed in the Fall of 1992 and included:

- a.* Installation, maintenance, and removal of upstream sheet-pile cofferdam and downstream stop logs.
- b.* Removal of deteriorated concrete on lock chamber walls at monoliths number 4 through 22 and 35 through 53 above el -2 and replacement with precast concrete stay-in-place panels and concrete infill.
- c.* Removal of middle gates and deteriorated concrete at gate locations and filling with cast-in-place concrete below el -2 and with precast panels above el -2.
- d.* Removal of loose concrete on the river wall monoliths number 39 through 55 above el -2 and monoliths 32 through 38 above el 12 and installation of new precast concrete panel jacketing with concrete infill.
- e.* Removal of deteriorated surface concrete on apron areas and replacement with cast-in-place concrete, including wall cap structure.

- f.* Installation of 6 new ladders and 10 new line poles in the lock chamber.
- g.* Resetting of existing snubbing posts.
- h.* Installation of new railings to replace deteriorated railings.
- i.* Repair of culvert valve tracks.

Revisions from Lock 22 precast concrete stay-in-place forming systems used in the Concrete Repair Program at Troy Lock included:

- a.* 7-1/2-in.-thick panels used in the lock chamber and 5-1/2-in.-thick panels used on the river walls, compared to 6-1/2-in. panels at Lock 22.
- b.* 4-1/2-in. minimum infill concrete, compared to 5-1/2-in. minimum at Lock 22.
- c.* Two rows of 11-ft, 10-in.-high panels for the lock chamber, compared to one 10-1/2-ft-high panel at Lock 22.
- d.* Weld plates utilized at Lock 22 were eliminated and embedded rebar panel hoops for anchorage used on rear face of panels.
- e.* Jack screw system for leveling utilized at Lock 22 was eliminated and recessed shim plates used.
- f.* Controlled backfilling of infill concrete limited pours to one-half panel height, compared to continuous operation at Lock 22.
- g.* Flyash used in cement to improve strength and impermeability of panels at Troy, but not at Lock 22.
- h.* Design based on allowable crack width of 0.006 in. versus 0.01 in. at Lock 22.
- i.* 1-in. chamfer in 12 in. was added on the lock chamber panels either side of the vertical joints. See Plate A-14.
- j.* Revised erection procedures utilizing two erection anchors at the top of each panel for vertical lifting and setting and the anchorage hoops on the rear face for handling and unloading, in lieu of only the lifting strands on the back of panels utilized at Lock 22.
- k.* Setting of panels utilized laser equipment for control of vertical and horizontal alignment. Additional adjustments for the lock chamber panels were provided using temporary holding and adjustment brackets.

- l.* Vertical joints were left open at Troy due to the porous nature of the existing concrete substate and the existing leaking monolith joints. Infill concrete was saw cut at vertical joints and filled at the bottom horizontal joints. Caulking of vertical joints with urethane sealant as performed at Lock 22 was not utilized at Troy, since it could result in the development of hydrostatic pressure behind the precast panels.

Concrete Repair at Lock 0-6, Oswego, NY

The rehabilitation of Lock 0-6 on the Oswego River outside the City of Oswego, NY, for the New York State Department of Transportation (NYSDOT) Waterways Maintenance is presently under construction using precast panel technology similar to that used at Troy Lock. The work includes the use of precast panels in the lock chamber and at the waterline along the river wall. Construction at Lock 0-6 began in early 1992, but the fabrication and installation of precast panels is not scheduled until the 1992-1993 winter work season.

Although the design and detailing of the precast panels at Lock 0-6 followed closely to that used for Troy Lock, there are some differences which include:

- a.* Lock 0-6 has three rows of 9-ft-high panels in the lock chamber and one row of 4-ft-high panels on the river wall.
- b.* Unlike Troy Lock, fabrication of the panels at Lock 0-6 may be performed by the prime Contractor in the field. Use of a certified precaster is not a requirement.
- c.* Since there is very little seepage through vertical monolith joints at Lock 0-6, joints will be sealed with open-cell foam with weep openings left at the bottom only.

3 Precast Panel Design and Detailing

Design Criteria

Load cases

The lock chamber panels were subjected to two major load types. These were the handling loads and the infill concrete placement loads. The following is a description of each one of these loads.

a. Handling of panels:

- (1) Lifting of panel from horizontal casting bed.
- (2) Lifting of panel for horizontal truck loading.
- (3) Lifting of panel for vertical installation.
- (4) Tilt-up installation - Panel supported on bottom.

b. Placement of infill concrete:

- (1) Placing of lift no. 1 (bottom half of panel).
- (2) Placing of lift no. 2 (top half of panel).

These loads were then evaluated and used for the design of the precast panels. The design methods are outlined in the following paragraphs.

Allowable stresses

Working stress design methods were employed for this rehabilitation project. The allowable working stresses used for the precast concrete design were in accordance with EM 1110-1-2101 (Headquarters, U.S. Army Corps of Engineers, 1963). These stresses were:

$$f_c = 0.35f'_c = 2,450 \text{ psi} \quad (f'_c = 7,000 \text{ psi})$$

$$f_s = 20 \text{ ksi} \quad (f_y = 60 \text{ ksi})$$

Crack control

Serviceability requirements were specified to aid in the prevention of steel reinforcement corrosion in all panels. The maximum crack width, as calculated by the Gergely-Lutz equation (American Concrete Institute (ACI) 318-89 (1989a)) was limited to 0.006 in. on the exposed (front) face. This limit was in accordance with the recommendation of ACI 224 (ACI 1989b) for concrete exposed to seawater spray or wetting and drying cycles. For temporary handling loads, the limiting crack width on the exposed face was increased to 0.010-in. Crack widths on the back face were not restricted.

Nominal steel reinforcement (wire mesh) was provided in the back face of river wall panels, in addition to the single layer of design reinforcement, to provide additional crack control.

Concrete mixture proportions

The concrete mixtures for the Troy Lock project were an important factor in determining the quality and serviceability of the finished product. The concrete mixture properties specified were:

a. Precast concrete: (See Appendix RBS)

$f'_c = 7,000 \text{ psi @ 28 days}$

Entrained air = 5 to 7%

Maximum water/cement ratio = 0.40

Maximum aggregate size = 1-1/2-in. nominal

Silica Fume = 7% by weight of cement maximum (optional)¹

Coarse aggregate hardness = Mohs hardness > 5

b. Infill concrete:

$f'_c = 3,000 \text{ psi @ 28 days}$

Entrained air = 4 to 7%

Maximum water/cement ratio = 0.45

Maximum aggregate size = 3/4-in. Nominal

Slump = 1 in. to 4 in., except for chemical admixtures - maximum 8 in.

¹ Type C Flyash was substituted for silica fume by the precast plant.

Design methods

The design of the precast concrete panels was performed using both a finite element model (STAAD - III) and conventional concrete design methods. The design procedures used for the various loading conditions are summarized as follows:

- a. *Handling of panels.* The precast panels were designed using a four-point lifting system. For the typical size (20- by 12-ft) panels, pick points were set in each corner. For the various panel sizes, most efficient lifting arrangements were identified and detailed in the drawings. These points were used as support locations in the STAAD - III finite element analysis. Design moments were then obtained and required reinforcement was designed accordingly. The precast panels were specified to be moist cured until concrete reached a strength of $0.20 f'_c$. The supplier then had the option of lifting the panels to a storage location by an approved lifting method. During the entire shop handling procedures, crack widths were limited to 0.010 in. The panels were also modeled, in a similar manner, for the anticipated tilt-up installation procedure.
- b. *Infill concrete placement.* Following the precast panel installation, the infill concrete is placed behind the panels. The design form pressures were assumed to increase linearly with a concrete density of 150 pcf. The infill concrete lift height was limited to 6 ft to minimize the actual load applied to the panels. The design approach assumed one-way slab action spanning vertically between top and bottom Rshe-boltS form anchors at an assumed horizontal spacing of 4 ft on center. The reinforcement was designed using working stress procedures and checked against serviceability (cracking) requirements. Reinforcement design was generally controlled by serviceability criteria.
- c. *Special panels (i.e. ladder and line pole panels).* The loads for the special panels were similar to those listed in the previous two sections. The design procedure assumed a simple span between two rectangular beams. The reinforcement was designed using working stress design procedures for all handling and placement loads and checked against serviceability criteria.
- d. *Erection of panels.* The primary concern in the erection procedure was the shear capacity of both the lifting loops on the back of each panel and the erection anchors in the top of each panel. These capacities controlled the overall dimensions of the precast panels. The precast concrete supplier was required to submit shop drawings for approval, which indicated the specific handling attachments.
- e. *Dowel anchors.* The maximum dowel anchor spacing of 4 ft was provided in accordance with recommendations of Liu and Holland (1981). The spacing recommendation from this report is based on anchorage of cast-in-place repairs; however, since resulting forces on

completed repairs are believed to be similar for cast-in-place and precast methods, the report was considered applicable to either method.

- f. Precast panel details are shown in Plates A-1 through A-14, Appendix A.

Size restrictions

Panel dimensions (Plates 1 and 2) were arrived at considering economics and safety of fabrication, lifting, erection, and transportation costs associated with overwidth vehicles. Lifting of the panel during installation was by a single crane line attached to two 8-ton erection anchors in the top edge of each panel. Minimum panel thickness required to attain the nominal 8-ton capacity of typical erection anchors is 7-1/2 in. Panel height was limited to 11 ft, 11 in. to avoid additional transportation costs of overwidth loads. Maximum panel length was limited to 28 ft, 6 in. so as not to exceed the safe loading capacity of two anchors per panel. This results in the ability to cover the widths of a majority of the monoliths with a single panel, thereby minimizing vertical joints.

Supplier restrictions

To ensure the qualifications of the precast plant, contract specifications required that the manufacturer of the panels have a minimum of 5 years experience in producing similar panels, have an established quality control program in effective operation in the plant, and be currently certified by the Prestressed Concrete Institute (PCI) or the National Precast Concrete Association (NPCA). Also, an automated batch plant approved by NYSDOT was required.

Aggregate source restrictions

Aggregates were required by specifications to meet the requirements of American Society for Testing and Materials (ASTM) C 33 (ASTM 1990) and be provided by an approved NYSDOT source. Coarse aggregate with a Mohs hardness of 5 or greater was specified for improved panel abrasion resistance.

Panel Configurations

Lock chamber panels at Troy Lock, as shown in elevation in Plate #1, are 11 ft, 10 in. high and placed in two vertical rows. The typical panel length is 20 ft, but panels vary in exact size from approximately 21 ft to 6 ft to align with existing lock wall monolith joints.

River wall panels, as shown in elevations in Plate #2, are installed in three rows with the bottom row panels being 5 ft high and the middle and top row panels being each 10 ft high. The 5-ft panel was placed at the bottom of repair for constructability reasons in the tidal area. The typical panel length is 20 ft, but panels also vary in exact size from 23 ft to 13 ft, 8 in. to align with monolith joints. Sloping transition wall areas have parallelogram shaped areas done in cast-in-place methods, in lieu of costly special precast forming.

Special panels are provided in the lock chamber at four line pole locations and at six combination ladder and line pole locations. The panels are 6 ft long by 11 ft, 10 in. high for line poles and 8 ft long by 11 ft, 10 in. high for combination ladders and line poles. The center of each panel is recessed 1 ft, 1 in. to accommodate ladders and line poles. Special panels are shown in Plates A-8, A-9, A-10, and A-11, Appendix A.

Panel thickness varies for the different types of panel. Lock chamber panels are 7-1/2 in. thick, river wall panels are 5-1/2 in. thick, and special lock panels are 1 ft, 8-1/2 in. thick, except at the recesses where thickness is reduced to 7-1/2 in. Typical panel sections are shown in Plates #3 and 4 for the land wall and river wall, respectively.

Lock chamber panels have a 1-in. by 12-in. taper and a 1-in. chamfer along the vertical joints to reduce impact spalling at the joint locations. Special panels have a 1 in., two-way taper and 1 in. chamfer on each end of the panels. River wall panels have a 1 in. chamfer on all exposed edges. These tapers and chamfers are shown in Plate A-14, Appendix A.

Panel Details

Reinforcing bar details vary for the different types of panels. Lock chamber panels, as shown on Plate A-2, Appendix A, have #5 bars at 6 in. on center each way on the front face and #5 bars at 12 in. on center each way on the rear face. A 2-in. concrete cover is provided on all faces.

River wall panels, as shown on Plate A-4, Appendix A, have one layer of #5 bars at 6 in. on center each way located at 2 in. clear from the front face, plus a layer of 6 by 6 - W4 by W4 welded wire fabric at 3/4 in. clear from the rear face.

Special panels in the lock chamber, as shown on Plates A-9 and A-11, Appendix A, have similar reinforcing, as noted for lock chamber panels (#5 at 6 in. each way front face and #5 at 12 in. each way rear face) with additional bends and splices as shown.

Anchor details are similar for all panels, although the vertical spacing does vary. Panel anchors consist of a #6 hoop-shaped rebar which extends 4 in. from the rear face of panel. Anchors are located at a 4-ft horizontal spacing (to match dowel spacing) and at vertical spacings ranging from 3 ft, 6 in. to 5 ft, 4-in.

Shebolt form anchors are used to hold the panels in position while the infill concrete is being poured. Shebolts are located at 4-ft centers above and below lock chamber panels and through the top and bottom of river wall panels. Shebolts are designed to withstand forming pressures and can be used to adjust panel alignment. Rebar and form anchor details are shown in Plate A-13, Appendix A.

Lifting details. Two 8-ton erection anchors are embedded in the top of each precast panel to facilitate lifting. Panel anchor bars are also utilized for lifting from forming beds, storage yard or shipping trucks, or as necessary for tilting into setting position on the walls. Locations of erection anchors are shown in Plates A-1, 3, 5, 8, 10, Appendix A.

Leveling/adjustment details. The design assumed that vertical and horizontal adjustment would be performed using the shebolt form anchors and recessed bottom shim plates where required.

Joint details vary for the lock chamber panels and the river wall panels. Lock chamber panel layout includes 1/2-in. vertical joints between adjacent panels and between panels and cast-in-place concrete, and 2-in. horizontal joints below each row of panels. The 2-in. horizontal joints allow for the installation of the shebolt form anchors. The vertical joints are left open for drainage and the horizontal joints are filled by infill concrete placement.

River wall panel layout includes 1/2-in.-vertical joints and 1/2-in.-wide by 1-in.- deep horizontal joints. The vertical and horizontal joints in walls above the tidal zone are sealed with backer rod and 1/2-in. joint sealant. Within the tidal zone, the horizontal joints are grouted and the vertical joints left open due to the inability of flexible sealants to cure within the time available before being exposed to moisture. Joint details are shown in Plate A-14, Appendix A.

Cap details. The top 2 ft of lock chamber walls and the top 4 ft of river wall are designed for reinforced, cast-in-place concrete replacement with anchors doweled into the existing wall structure (Plates 3 and 4). Cast-in-place concrete was selected because of the irregular shapes required.

4 Precast Panel Production

Production Facilities

Precast panels for the lock chamber and river wall were produced in late 1991 and early 1992 at The Fort Miller Company plant, Schuylerville, NY, 20 miles north of Troy Lock.

All panels were fabricated in a insulated building kept at 70 °F by use of gas-fired infrared heaters. The building was serviced by a 10-ton gantry crane which was used to move the panels.

Concrete used in the panels was manufactured in a NYSDOT approved automated batch plant (Figure 3 and 4). The plant was equipped to heat both the aggregates and the water. The temperature of the freshly batched concrete was regulated by controlling the temperature of the mix water. All batching was computer controlled and monitored by a computer generated batch ticket which indicated the actual amount of each material used as well as the water/cement ratio achieved.

Concrete was transported from the batch plant to the production facility with a standard concrete truck. Immediately upon arrival at the production facility, inspectors from the Quality Control Department performed the required tests.

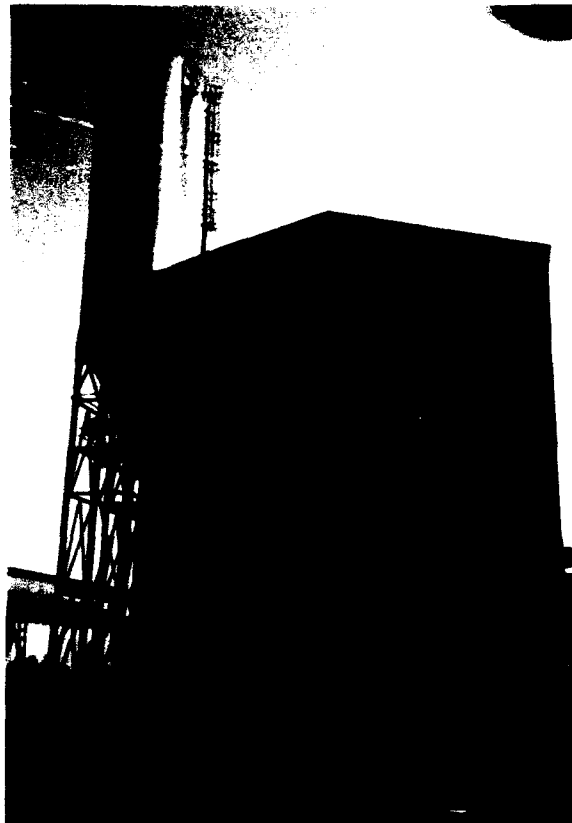


Figure 3. Automated batching plant

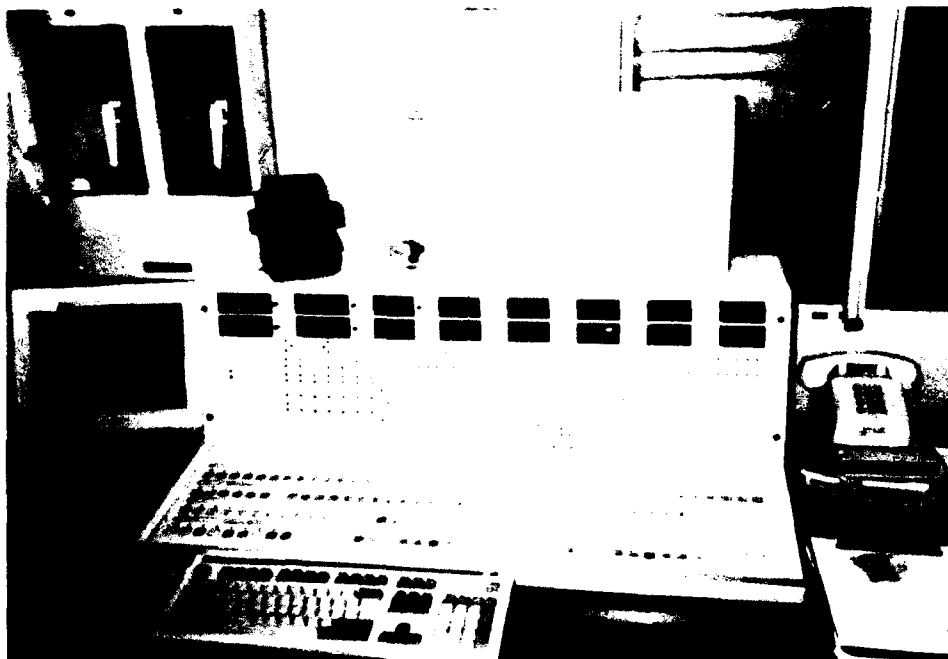


Figure 4. Batching plant computerized controls

Production Schedule

Steps in the panel production are as follows:

- a.* Clean form table and side rails
- b.* Install side rails
- c.* Lay out and install embedments
- d.* Lay out and install reinforcing steel
- e.* Check all diagonals of form for accuracy
- f.* Double check all embedments and bars
- g.* Place concrete in mold and consolidate by bed vibration
- h.* Rake finish
- i.* Cover with wet burlene for overnight cure
- j.* Strip forms next day
- k.* Remove panel to wet cure area

1. Begin with step a again

Timing of production followed this schedule:

- a. Stripped molds at 6:00 a.m.
- b. Setup followed immediately
- c. Poured molds from 10:00 to 12:00 a.m.
- d. Finished and covered panels by 2:30 p.m.

Concrete Mixture

Mixture proportions for the 7,000-psi concrete mixture were as follows:

	Mass/cy (Saturated Surface-Dry)	Volume (CF)
Type I/II Cement, lb (Brand A)	376	1.91
Type I/II Cement, lb (Brand B)	376	1.91
Mineral Admixture 2, lb	54	0.34
Aggregate NYSDOT Test #90AF99, lb	1,008	6.21
Aggregate NYSDOT Test #90AG39C, lb	1,815	10.77
Water, lb (Gal. - US)	246 (29.5)	3.94
Air entrainment, %	<u>7.0</u>	<u>1.89</u>
Total		26.97
Low-range water reducer, oz-US	23.0	
High-range water reducer, oz-US	119.9	
Air-entrainment admixture, oz-US	12.1	
Water/cement ratio, lb/lb	0.31	
Slump, in.	7.50	
Concrete unit weight, pcf	143.6	

Revisions from contract specifications were:

- a. The original specifications called for the optional use of silica fume to achieve 7,000-psi design strength. The precaster opted to use Type C Flyash instead, based upon prior experience.
- b. The original specifications required cores from finished panels to verify conformance to air specifications. The precaster requested that this requirement be dropped, because it would have resulted in unsightly patching. The precaster instead submitted historical data demonstrating good correlation between plastic air tests and linear-traverse tests made on the same concrete mix design. The use of air tests on plastic concrete was approved based upon this good correlation. Also, freeze-thaw durability (ASTM C 666 (ASTM 1991)) tests were performed at an independent laboratory. Cores were removed from test panels only and tests were performed as required by the contract specifications. Additional samples were taken at the batch plant for compressive strength and air void tests.

Formwork

Casting tables were made which consisted of a grid of steel channels supported by a heavier framework of steel wide flange sections which were in turn supported by concrete piers (Figure 5). This resulted in a crawl space

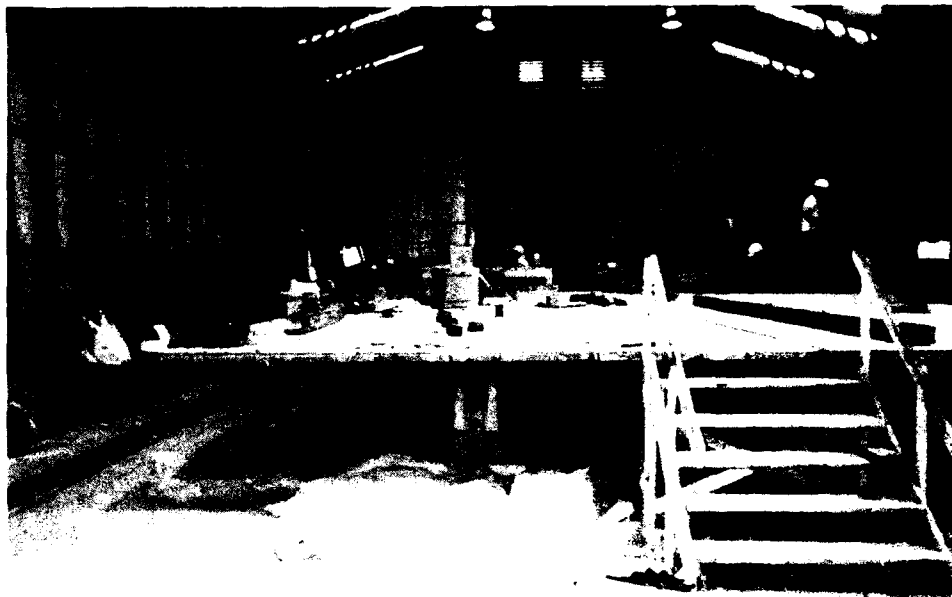


Figure 5. Forming table being prepared

under the forms facilitating installation of embedments. The steel channels supported a 3/4-in. plywood deck which was faced with formica. Side rails, used to form the edges of the panel, were made out of dimension lumber and formica faced plywood. These were, in turn, bolted to the deck as required. In each bed, larger panels were made first, then followed by successively smaller ones. This maintained a high-quality finish while facilitating bolting of side rails. Finally, special 4-ft by 4-ft frames spanned the form. These frames supported the anchor loop which protruded from the back of the panel.

Reinforcing and Inserts

Proper placement was necessary:

- a.* All reinforcement was uncoated and placed in the position indicated on the contract drawings (Figure 6).
- b.* Inserts were carefully laid out as indicated on the approved shop drawings for each panel.
- c.* Inserts were attached by bolting them to the form with specially designed "older" hardware.



Figure 6. Form and reinforcing steel ready to pour

Revisions from contract drawings were as follows:

- a. All inserts were installed as indicated on the contract drawings, except the diameter of the shebolt holes for the river wall panels was increased from 1-3/4 in. to a 3-in. tapered hole to provide construction tolerance for installation of dowels and panels. Additional inserts were installed which allowed the contractor to use leveling bolts. These bolts penetrated the panel and extended to the monolith. By adjusting these bolts, an exact plane on the finished side of the panels could be achieved.
- b. Other inserts were added to allow the contractor to "hang" the panel in the proper location until infill concrete was placed.

Casting Operation

Concrete was placed with a concrete bucket which was maneuvered by the gantry crane (Chapter 4).

- a. *Vibration methods:* External vibrators were mounted on the steel frame work underneath the form. These were used for most of the consolidation. Additional internal vibration was used in the corners of the panel as required.
- b. *Finishing:* After the concrete was fully vibrated, it was finished with a wood float. Finally, the concrete was raked to give it a rough texture to enhance bonding to the infill concrete.

Curing Operation

After the finishing operation was completed, the panels were covered with wet burlene for wet curing. After the panels were stripped from the form, they were stacked, one upon the other, covered again with wet burlene and finally covered with polyethylene to prevent moisture loss (Figure 7).

The panels were cured overnight in the molds as previously described. In the final curing operation, they were stored for 5 to 7 days until such time as design strength had been reached.



Figure 7. Panel curing after removal from table

Quality Control Procedures

Tolerances

Prepour checks: Dimensional tolerances were achieved by conducting two prepour dimensional checks. These checks were performed by a quality control inspector working together with the foreman in charge of production.

Postpour check: A postpour check was conducted at stripping time by the inspector and foreman. All results were recorded.

Mixing

All batching and mixing operations were monitored by the Quality Control (QC) Department. A computer-generated print-out of each batch was reviewed by QC technicians. A quality control technician also performed the required plastic tests at the casting site. No retempering of any concrete truck was allowed.

Temperature control

The temperature of the freshly mixed concrete was first checked at the batch plant where adjustments were made on the next batch, if necessary.

The temperature of the concrete was again checked at the casting site and any required changes were radioed to the batch plant.

Testing

Slump, air, and temperature tests were performed at the casting site as required by the specifications and the approved shop drawing. Extra cylinders were taken to allow early compression testing of the hardened concrete. This was also indicated on the approved shop drawings. Yield tests were also performed. See Figures 8 and 9 for compression testing apparatus and water curing of cylinders.



Figure 8. Compression test of concrete cylinders

Handling, Storage, and Shipment

When the production of the panels was completed, these steps were followed:

- a. Panels were removed from the casting tables by the gantry crane (described in Chapter 4).
- b. Panels were stored in the casting building one on top of the other under plastic cover until design strengths had been reached.



Figure 9. Water curing of concrete cylinders

- c. The panels, upon reaching design strength, were loaded on a yard truck which transported to the yard storage area (Figure 10).
- d. The panels were unloaded from the yard truck and stacked with a yard crane at the yard storage area. Care was taken to place dunnage under the product to prevent cracking. Panels were shipped flat and, in general, two panels were shipped per load.

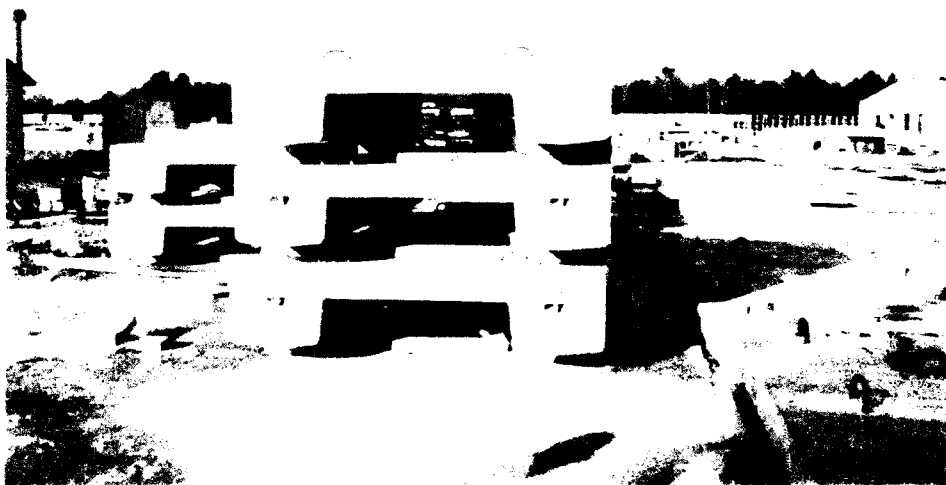


Figure 10. Panel storage in precast plant yard

5 Lock Preparation

Concrete Removal

Removal of lock wall surface concrete was performed by subcontractor PRK Drilling and Blasting, Inc., Cambridge, NY, using a combination of drilling with blasting and grinder machinery. A minimum of 12 in. of surface concrete was removed, with additional depths required at ladder and line pole locations (2-ft depth) and at localized more deeply deteriorated wall areas as required.

Removal of concrete on the river face of the river wall was limited to that required to install the new concrete cap structure (roughly 4 ft high by 2 ft deep) and to remove any localized loose or deteriorated concrete. Removal was performed using jackhammers and hand tools.

Sawcutting was used around the perimeter of areas to be removed by blasting. A 10-in.-deep sawcut was made horizontally below all removal areas at el -2.0 to provide the finish ledge for precast panel placement.

Line drilling and blasting were performed using an approved drilling and blasting plan in accordance with the contract specifications which include 3- to 3-1/2-in.-diam vertical holes drilled on 1-ft centers down to el -1.5. Holes were drilled with centers at 14 in. from the original local wall surface to provide the required 12-in. minimum concrete removal. An Austin 150-grain detonating cord was used. An average daily production rate of 21 holes drilled using five drill rigs and two to four blasts of six holes each was achieved.

Blast monitoring was performed using either a Berger SSU-2000 DK or Vibr-Tape G.MS4 calibrated unit. All blast vibrations were below the specific maximum of 2 ips.

Additional removal in the lock chamber was performed at localized areas which suffered incomplete removal by blasting at ladder-line pole areas, at areas of increased depth of deterioration, and at perimeter areas using a Mitsui concrete grinder mounted on an Akerman H14BLC tracked hoe, supplemented with jackhammers and hand tools as necessary.

Surface Preparation and Cleanup

The Contractor requested a waiver for cleaning the lock chamber walls after removal of deteriorated concrete. His request to substitute high-pressure air in lieu of water to eliminate icing of the walls and scaffolding prior to installation of the precast panels was approved. Following sandblasting of existing concrete surfaces, including the area behind the removed middle gates, a final air jet cleaning was performed utilizing a Clemco model SCWB 1648 blast machine with 100-psi air pressure from a 3/4-in. wand for both the lock chamber and river wall surfaces. All debris on the lock chamber floor was removed using backhoes, dumpsters, and the land side cranes.

Anchor Installation

The anchor dowels used for both the lock chamber wall and the river wall were No. 6 deformed reinforcing bars drilled and grouted into the existing wall structure. The dowels were installed to line up with the hoop panel anchors at a spacing of 3 to 5 ft in each direction. Locations of panel anchors and dowels are shown on elevations in Plates A-1, 3, 5, Appendix A.

An adjustable steel drilling template was fabricated for the typical monolith section to maintain proper dowel locations (Figure 11). The template included



Figure 11. Anchor template set in position on lock wall

a steel drill guide which was welded to the template to achieve the required 10-deg drill angle for the anchors. The template was lowered into proper location by crane and fastened with expansion anchors to temporarily hold in place during drilling operations.

The contractor utilized a two-part epoxy bonding system as specified for the anchor system, which was installed in 7/8-in.-diam, 1-ft, 5-in.-deep drilled holes for the lock chamber and in 7/8-in.-diam, 2-ft, 5-in.-deep holes for the river wall (shown on contract details in Appendix A). The listed ultimate tension capacity of the epoxy anchors is 38,900 lb.

Pullout testing was performed on roughly 5 percent of the anchors installed using a fabricated three-legged frame and jacking to 21,000 lb minimum (working stress capacity of the anchors). The pullout tests on the bent bars (dowels) had to be performed with a specially designed rig for pulling the bars axially while not destroying the dowels. Several of the initial anchors did not successfully pass the pullout test due to improper mixing of the components or inadequate setup. Those anchors were reinstalled and retested.

6 Panel Installation

Procedures and Equipment

Precast panels for both the lock chamber and the river wall were lifted off the delivery trucks using a crane located just east of the lock chamber and placed on rubber tires on the ground in a horizontal position. Precast panels were lifted from the horizontal position on the truck using lifting inserts at each corner (see Plate A-1, Appendix A, for locations). After panels were set on the tires, the lifting inserts were removed and the crane was connected to the erection anchors. The panels were then raised to a vertical position while supported by the rubber tires on the bottom edge. The panels then were lowered into their final position using the 8-ton erection anchors mounted on the top edge of the panels (Figure 12).

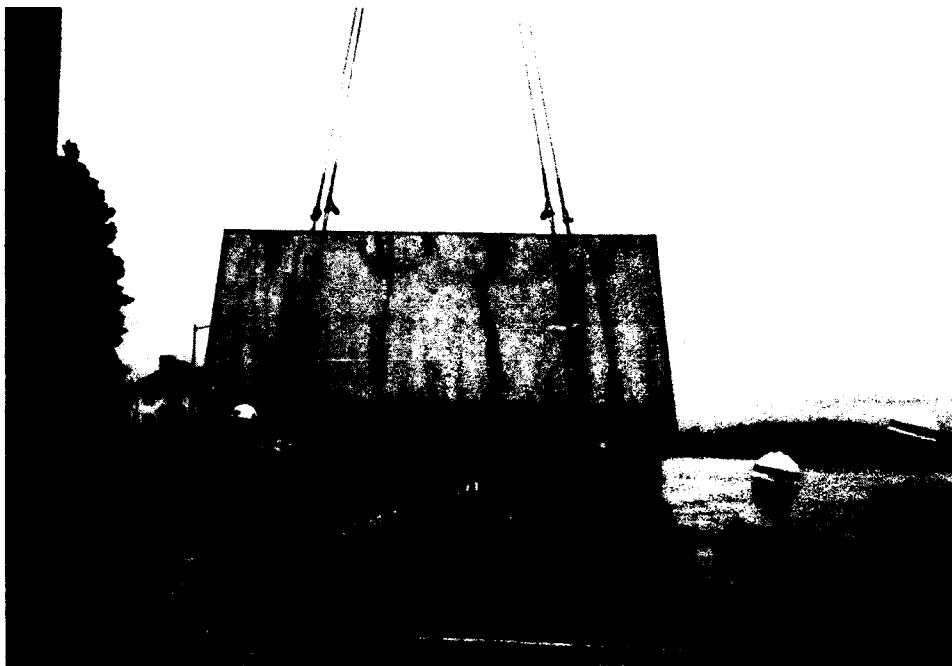


Figure 12. Lifting river wall panel into position

River wall panels were lifted similarly into position using the top erection anchors and the lower lifting inserts (see Plates A-3 and A-5, Appendix A and Figure 13 for insert locations). The Contractor fabricated a template to match the slope of the river wall and rigged the panel to approximate the template angle prior to swinging the lower river wall panels in place (see Figure 12). Final adjustments were made with panel in place. A floating barge was used for worker access and tool storage but was not required for erection purposes.

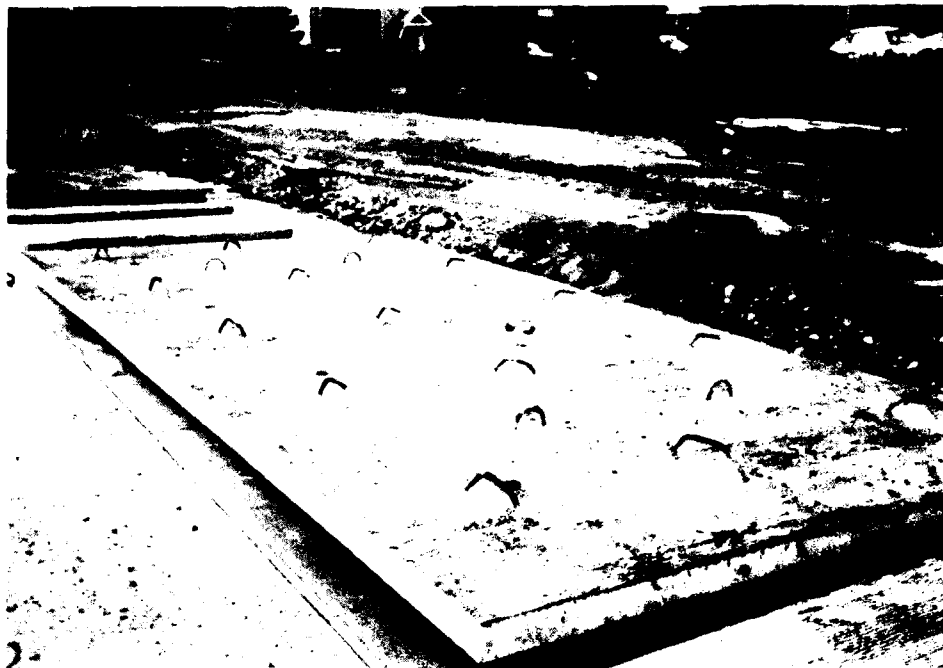


Figure 13. River wall panel stored east of lock

Panel Placement

Lock Chamber panels were placed in the winter work season of 1991-1992 when the lock was closed to shipping. The panels were lowered into place by crane onto steel shims placed on the horizontal shelf at el -2 or on top of a previously placed panel. The shims were set back a minimum of 2 in. from the panel face since they remained a part of the permanent structure. Panels were set in position using temporary holding and adjustment brackets, shebolt form anchors above and below the panels at roughly 4-ft spacings and vertical steel double channel strongbacks spanning between the protruding shebolts (see Figures 14, 15, 16). The strongbacks were added by the Contractor during the design approval process for the forms, to provide additional support during concrete infill placement.

River wall panels were installed in the late Spring and Summer of 1992 while the lock was in operation. The panels were lowered into place by crane



Figure 14. Bottom row of lock panels with strongbacks and scaffolding



Figure 15. Top of lower precast panel prior to infill placement



Figure 16. Top of lower lock panel after infill placement

to the bottom position or on top of previously placed panels (see Figure 17). Panels were set in position using adjusting bolts and shebolt form anchors through each panel. Even though the Contractor scheduled panel installation for low tide, the bottom row of panels was set 1 to 3 ft underwater eliminating the need for costly cofferdams.

All final vertical and horizontal alignment of panels was controlled by use of an alignment laser.

Bulkheads used for concrete infill placement for the lock chamber panels consisted of a 2 by 10-in. wood strip spanning horizontally between shebolt anchors along the 2-in bottom joint. Bulkheads on the river wall panel were steel plates varied up to 36 in. to fill in the large voids found in the concrete near the lower portion of the repair section caused by erosion. In addition, the Contractor had to fabricate an adjustable bulkhead to provide for the irregular surface. See Plate No. A-12, Appendix A for closure form.

Due to the anticipated seepage through the vertical monolith joints, it was decided to leave the vertical joints for all panels open. However, to prevent leakage of infill concrete from these joints, a continuous foam backer rod was installed after the panels were set and prior to infill concrete placement. (Plate A-13, Appendix A).



Figure 17. Setting river wall panel

Reinforcing Steel Placement

Placement of panels was performed to ensure that panel anchors lined up with hooked dowel bars drilled into the existing wall monoliths. Once the panels were adjusted into final position, vertical # 6 reinforcing steel bars were manually installed from the top down to intersect the panel anchor hoop and the dowel bar hook. When the bar was properly located, it was tied to the dowel hook to keep from becoming misaligned during infill concrete placement.

Infill Concrete

Concrete mixture requirements for the infill concrete are noted on page 10 of this report. Concrete was air entrained with a minimum 3,000-psi compressive strength and maximum of 3/4-in. aggregate.

- a. Placement of infill concrete in the 4-1/2-in. minimum thickness space behind the precast panels was performed using a concrete bucket lifted by the erection crane and a flexible elephant trunk (Figure 18). Concrete was placed continuously behind the row of panels for one-half of the panel height in one pour, except at the 5-ft-river wall panels where the full height was poured. After vibration and setting (4-hr minimum) of the infill concrete, the upper half was placed. Thickness of infill concrete was much greater at eroded waterline areas for the river wall

panels and at the middle gate areas in the lock chamber, where infill varied up to 36 in.



Figure 18. Infill concrete placement and vibration on river wall

- b. Vibration of the infill concrete was performed simultaneously with the placement operation using vibration rods.
- c. Underwater placement was required for the bottom pour on the lower river wall panel. An antiwashout admixture, Sikament 100 SC, was added to the concrete mixture for these pours. This allowed for the effective placement without providing additional cofferdaming.

Curing Procedures

The majority of lock chamber panels were set and infill concrete was placed between January 1st and mid-March 1992 when the ambient temperatures varied from 0 °F to 40 °F. Infill concrete was provided by the ready mix supplier at 65 °F or cooler. As was required by the specifications, adjacent air temperatures were maintained at 40 to 55 °F for the specified duration (15 days at 40 °F to 7 days at 55 °F) or until field cured concrete cylinders attained full design strength. The lock chamber was fully scaffolded from top to bottom and enclosed in plastic tarps (Figure 19). Panels were lowered down between wall surface and free-standing scaffolding removing the plastic tarps as necessary. The spaces between tarps and wall surface,

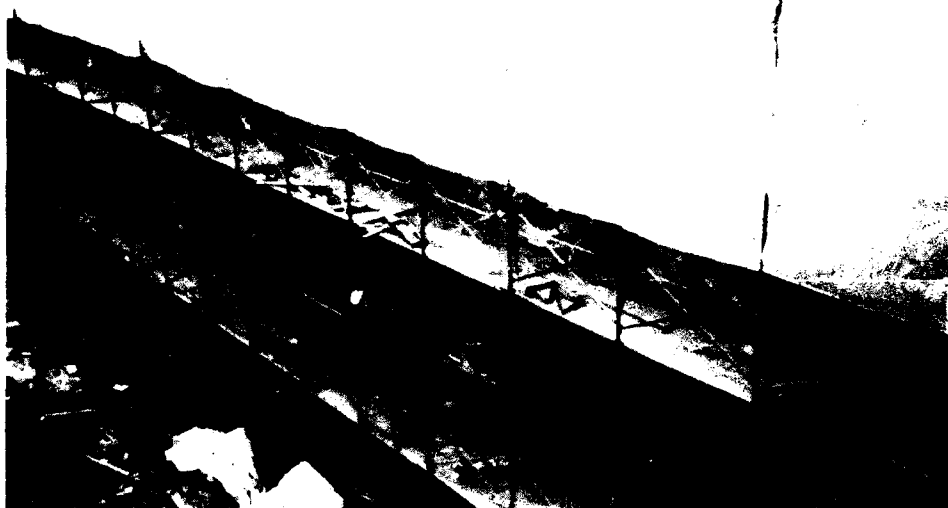


Figure 19. Tarp enclosure and scaffolding on lock chamber wall

including precast panels and infill concrete, was heated with propane "can" type heaters supplied by 500-gal tanks set in the lock chamber. Necessity of this heating procedure, however, requires further field verification and testing.

River wall panels were set and infill concrete was placed between June 1st and mid-August 1992 when ambient temperatures varied from 50 to 80 °F and no heating was required. Curing consisted of keeping the top exposed surface of infill concrete wet for a minimum of 7 days.

Quality Control Procedures

Panels were delivered to the site only when the walls were ready for their installation. Where possible, panels were unloaded and set into position immediately to avoid wasted handling time and the chance of cracking panels. However, many times the panels had to be stored on site, particularly the river wall panels which required installation of the adjustable bulkhead on the lower panels and special rigging. All panels were carefully checked for cracking prior to lifting off the delivery trucks and after setting into position. No cracking of panels was observed during this period for either lock chamber or river wall panels.

Proper alignment of panels prior to and after placement of infill concrete was verified using the laser equipment noted on page 30 and all panels were set with flush exterior surfaces. During infill concrete placement, concrete sampling and testing was performed by an independent testing firm to verify air content, temperature, slump, and compressive strength in accordance with

the specifications. Truck loads of concrete that did not meet the specified tolerances were rejected.

Finishing Operations

Joint preparation and repair

Foam backer rod, equal roughly in diameter to 1-1/2 times the joint space, was used in the vertical joints to hold back infill concrete during placement for both lock chamber and river wall panels. Following curing of the infill concrete for the lock chamber panels, the foam backer rod was removed and the infill concrete sawcut to full depth. Vertical joints were then left open to aid in the drainage of any seepage through the walls.

- a. After the infill concrete for the river wall panels had cured properly, a polyurethane elastomeric joint sealant was added to the vertical joints above the tidal zone. River wall vertical joints within the tidal zone were left open since a flexible sealant which would cure within the time period available between successive tides, could not be found.
- b. Horizontal joints between supporting shelf and panels and between rows of panels were patched with a concrete grout to fill shebolt holes and any honeycombing from infill concrete placement. At lock chamber horizontal joints some grinding was required to make the surface flush due to bowing of some of the wood bulkhead sections.

Crack repair

Following curing of the infill concrete, all 112 lock chamber panels and 65 river wall panels were again inspected for cracking. A total of 11 panels all in the lock chamber had some fine cracking. See Plates 5 and 6 for locations, widths, and patterns of cracking observed. As required by the contract specifications, cracks with widths > 0.006-in. (four locations) were repaired in place using an injection resin. Additionally, cracks < 0.006-in. (seven locations) were patched using a paste epoxy bonding agent, to seal the finer cracks.

Cast-in-place cap and apron

To seal off the top of the precast panels from seepage of water behind the panels, a reinforced concrete cap was placed on top of the lock chamber panels (2 ft high) and on top of the river wall panels (4 ft high). A 6-in. minimum apron slab was also added on top of the existing monolith wall to improve durability and reduce water penetration. Joints between panels and concrete cap and between cap and apron slab also received a 1/2-in. joint sealant. See Plates 3, 4, and A-13 for details.

7 Comparison of Quality: Precast Versus Cast-in- Place

Inspection Operations

Inspection of installed panels was performed by both the Albany Field Office QC staff and engineers from Bergmann Associates at various stages throughout the installation process. The primary means of access for inspection of the completed monolith panels was using the Contractor-installed scaffolding systems on both the lock chamber and river walls. Additionally, the panels were inspected by motor boat at various water levels after the lock was placed back in operation.

The primary purpose of the inspection operations was to check for possible cracking in the precast concrete panels. Cracks observed were measured using a magnifying crack comparator to determine crack widths. Cracks in panels were outlined, prior to repair, with a blue marker (see Figures 20 and 21).

Inspection of Precast Panels

Crack locations

Locations of cracks observed in the field are shown in Plates 5 and 6. There were 11 panels in the lock chamber that had fine cracks. Seven panels were located on the east wall (all except one being a lower panel) and four panels were located on the west wall (one upper panel). Most cracks were vertical starting from the bottom center of the panel and propagating upward. One ladder panel had horizontal cracks across the thinner midsection.

No river wall panels had cracks; however, two panels did have surface popouts at lifting insert locations in the lower corner.

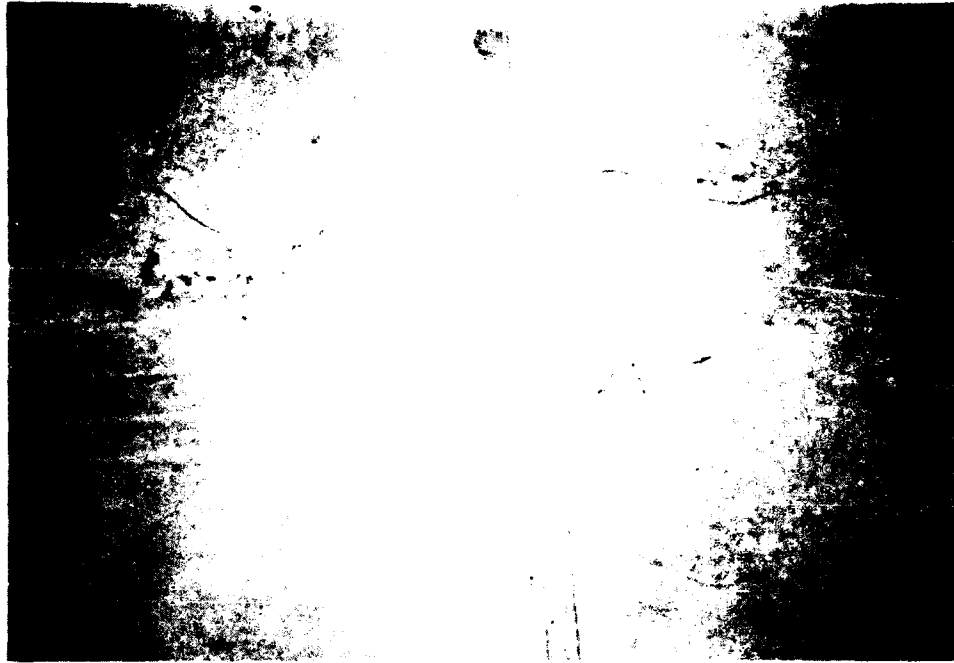


Figure 20. Outlined cracks in lock chamber monolith 4

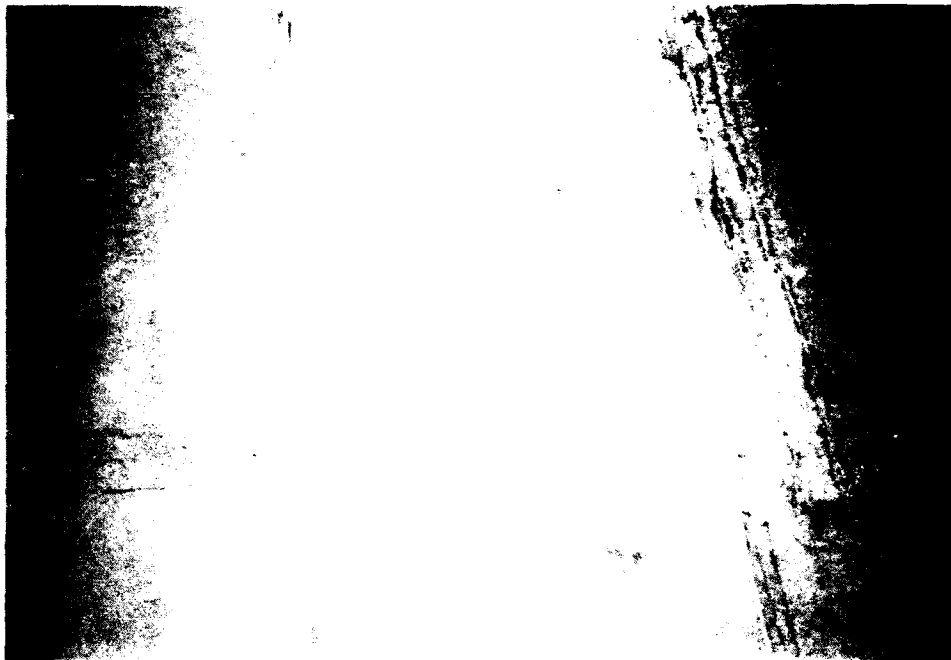


Figure 21. Outlined cracks in lock chamber monolith 5

Final inspection of panels also found some areas of crazing (numerous fine cracks on the surface concrete in hexagonal or octagonal patterns) on the lock chamber panels above the high water level.

Crack widths

Varied crack widths from 0.003 to 0.008 in. were determined using the crack comparator. These widths were considered fine and no obvious pattern of width existed.

Crack causes

It is speculated that the cracking observed in the 11 lock panels occurred during infill concrete placement, since no cracking was observed by the QC staff prior to that; although it is possible that some cracks may have occurred during placement of the panels. Most cracks were in the earlier installed panels and in areas between strongback supports.

Crazing observed in the upper areas of the lock panels was a result of the high heat of hydration of the curing panel concrete which creates fine surface cracking from shrinkage, especially in higher strength precast concrete. Since the crazing was not observed prior to completion of panel work, it is speculated that the heat from the sun on the areas above the waterline was the main factor.

Effect of cracks

Cracks have been repaired using pressure injection and surface sealing and, as such, are not expected to be detrimental to the performance or durability of the lock wall surfaces.

Crazing is also not expected to be detrimental to the panels, since testing of samples (taken prior to the observance of the crazing at final inspection) for freeze-thaw durability and compressive strength showed extremely good results.

Overall appearance

The appearance of precast panels in the lock chamber (see Figures 22 and 23) and on the river wall (see Figures 24 and 25) is very smooth and uniform. The limited cracking noted above is only visible at the locations repaired using the sealer material due to a difference in coloration from the concrete. Some seepage, especially in the west chamber wall, is noticeable at open vertical joints and at small openings in the horizontal joints.



Figure 22. Rehabed lock chamber returned to operation



Figure 23. Completed lock chamber panels

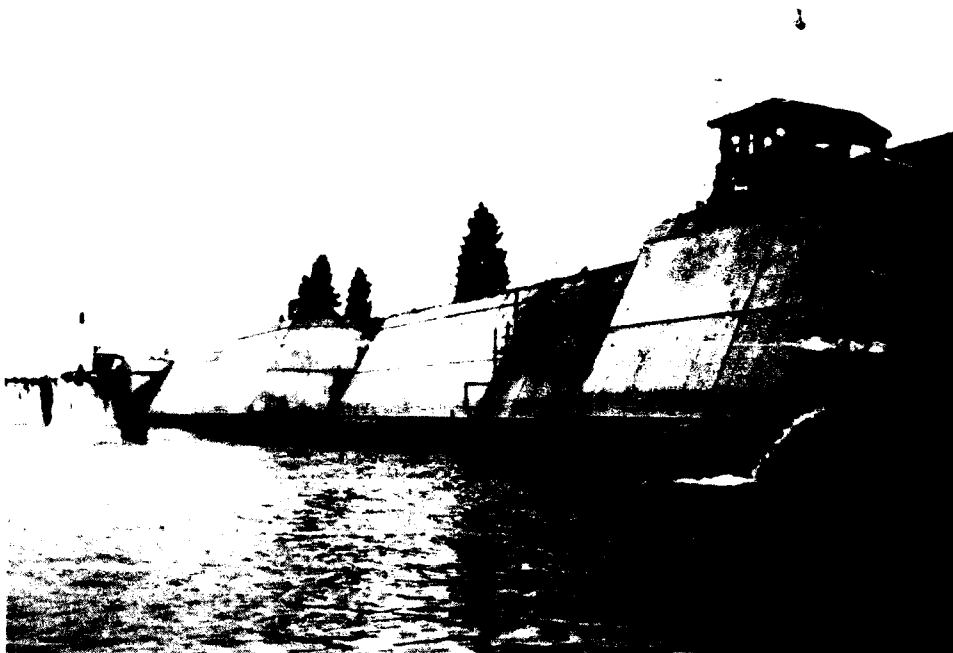


Figure 24. Rehab of river wall nearing completion

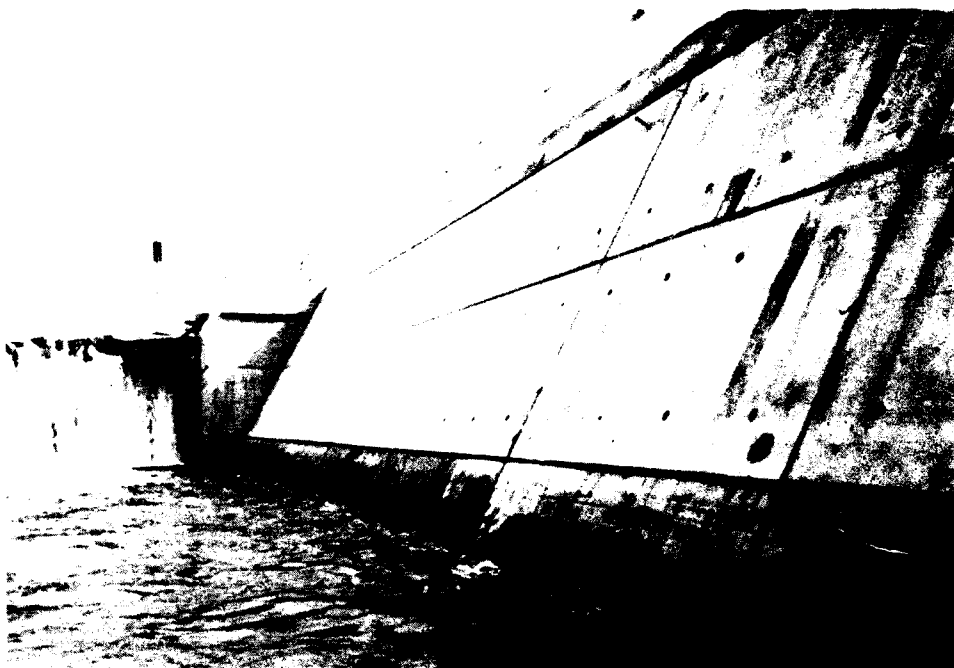


Figure 25. Precast panels installed on river wall

Inspection of Cast-in-Place Repair Areas

Crack locations

Inspection of the cast-in-place wall repair areas from both the Interim Repair (Phase I) and Concrete Repair (Phase II) programs shows extensive shrinkage cracking (see Figure 26). The cracking typically occurs horizontally and vertically in the middle of concrete pours with a variety of spacing from 2 to 10 ft.



Figure 26. Extensive cracking in cast-in-place repair adjacent to gate

Crack widths

Crack widths were not measured but are estimated in the 0.005-in. to 0.02-in. range.

Overall appearance

The appearance of cast-in-place rehabilitated wall areas is visually less attractive than precast repairs and gives the visual impression of concrete deterioration.

Original design

To reduce the effects of cracking in the cast-in-place repairs, the design for Phase I included a low water/cement ratio of 0.45 and a limiting placement temperature of 40 to 60 °F.

Durability Considerations

Precast concrete

Improved durability over cast-in-place concrete is expected from the precast wall panels based on the following factors:

- a.* High concrete compressive strength over 7,000 psi, verified by compression tests, resulting in high abrasion and impact resistance.
- b.* High freeze-thaw durability verified by laboratory testing.
- c.* Uniform air-entrainment of the concrete based on controlled precast plant conditions.
- d.* Very limited cracking and sealing of all observed cracks.
- e.* Improved surface density and reduced porosity based on low water-cement ratio, plant finishing, and curing.
- f.* Well detailed and accurately placed reinforcing steel to provide adequate concrete cover and limit future cracking.

Cast-in-place concrete

The expected durability of the newer cast-in-place concrete repairs should be improved over the previously existing concrete. However, due to the existence of significant cracking and lack of the improvement factors noted above for precast concrete panels, the durability is not expected to compare to that of the precast panels.

Schedule Considerations

Precast fabrication rates

Precast panels were produced at the precast plant at a typical rate of two panels per day. The rate is controlled by the size of the forming table, which at the Fort Miller plant allowed for two side-by-side panels. Panels were poured in a sequence to suit erection requirements in the lock and to limit yard storage in the plant where possible.

Precast installation rates

Approximately 100 precast panels of varying dimensions were installed in the lock chamber in approximately a 7-week period of time. The initial daily rate of panel installation, including setting, aligning, and anchoring the panels, was one or two panels per day due to the Contractor's learning curve. The installation rate improved up to as many as 10 panels being set in one day and an average rate of 3.8 panels per day was achieved for the 26 days in which panels were actually placed. A large percentage of the installation time was actually spent on installing the strongback supports and the wooden bulkheads, and the number set was also dependent on the number the contractor decided to set for each infill placement. Infill concrete placement was also performed within the same 7-week time period.

Approximately 70 precast panels were similarly installed on the river wall in approximately a 9-week period of time. Again, initial installation rates were only one or two panels per day. However, this was improved up to as many as eight panels in 1 day and an average rate of 3.0 panels per day was achieved during the 23 days in which panels were actually placed. Additional factors affecting installation rates on the river wall included the partial underwater placement of the bottom row of panels, adjustable bottom bulkhead plates, dependency of schedule on water levels of the river, and number of panels the contractor decided to set for each infill placement.

Cast-in-place productivity

The construction rates for the cast-in-place concrete wall repairs in both the Interim and Concrete Repair Contracts at Troy were highly variable. The cast-in-place work in the first contract was spread over two winters, while work in the latter contract was located in smaller areas throughout the work. Also the schedules were more dependent upon adjacent work and the weather conditions. However, with the typical forming and curing requirements of cast-in-place monolith placements, the rates should be considerably slower than for erection and infill of precast panels.

Finish Appearance

The finish appearance of the precast panels is excellent (see Figure 27). They are very smooth, uniform in color and neatly aligned. Their smooth surface provides for less accumulation of sediments and algae resulting in cleaner appearance. The cast-in-place concrete wall surfaces, on the other hand, with their extensive cracking, seepage lines, rougher texture, and less uniform coloring have a more deteriorated appearance.



Figure 27. Troy Lock and Dam, Rehabilitation almost complete

8 Economic Analysis of Precast Versus Cast-in-Place

Bid Prices

The following data summarizes the prices bid for concrete repair on recent lock chamber rehabilitation projects based on a total square foot cost of the repair. The prices were obtained from itemized bid tabulations and include work for existing concrete removal (cast-in-place removals are usually deeper, requiring thicker replacement section for NYSDOT locks), anchor systems, and replacement concrete.

Project	Bid Date	Approximate Bid Prices, Per sq. ft.			
		Cast-in-Place		Precast	
		Low ¹	Mean	Low ¹	Mean
Lock 22	1988	---	---	\$91	?
Troy	1988	\$24	\$44	---	---
	1991	---	---	\$33	\$40
Lock O-6	1992	---	---	\$49	\$45
Lock O-1	1991	\$42	\$59	---	---
Lock O-3	1992	\$32	\$47	---	---
Lock E-4	1990	\$40	\$42	---	---
Lock E-5	1990	\$48	\$46	---	---
¹ Low bid indicates price bid by awarded Contractor and is not necessarily lowest bid for these items.					

Comparison of Costs

The total square foot costs for lock chamber concrete rehabilitation are comparable for both the precast and cast-in-place methods. The average low and mean bids for the cast-in-place projects since 1988 are \$37 and \$48, respectively, per square foot; while the low and mean bids on Troy and Lock 0-6 for precast projects vary from \$33 to \$49 and from \$40 to \$45, respectively, per square foot. Since 1988 the average contracted price for both the cast-in-place repairs and the precast repairs at the various locks is equal at approximately \$41 per square foot, and the mean bid price at Troy for precast panels was roughly \$5 per square foot less than the mean bid price for cast-in-place concreting during the same period.

It is anticipated that as the number of qualified precast suppliers continues to grow and as contractors become more familiar with the efficiencies of precast panel use the costs of precast panel systems will be reduced.

9 Summary and Recommendations

Summary

It is an established fact that the quality of concrete produced in the controlled environment of a precast plant is superior to that which can be produced in the field, especially under the cold weather conditions found at Troy Lock. The QC procedures readily available at the precast plant allow for the production of a high-strength, abrasion resistant, and impermeable product that is expected to outlive cast-in-place construction.

Currently the costs of lock rehabilitation with cast-in-place concrete or precast panels are approximately equal. However, it should be noted that although the Troy project was constructed by a Contractor inexperienced in both lock rehabilitation work as well as the use of precast panels, the project progressed quite smoothly. Also, the efficiencies of using the precast panels became very obvious as the project was completed, and it is strongly believed that cost savings will be realized on future projects with this technology.

The appearance of the lock surfaces repaired using the precast panels is excellent. The smooth, uniform, virtually crack-free surface is a significant improvement over the cast-in-place resurfacing previously performed on most lock structures. The shrinkage cracking and subsequent water penetration, ensuing deterioration of embedded reinforcing steel, and continual maintenance problems associated with cast-in-place work are minimized by the precast product. Figures 22 and 23 show the completed lock chamber and Figures 24 and 25 show the riverwall panels after installation.

Compared with cast-in-place concrete, precasting offers a number of advantages including minimal cracking, durability, speed of construction, improved abrasion and impact resistance, reduced future maintenance costs, and improved facility appearance. Also, the precast product minimizes the impact of adverse weather.

Recommendations

The revisions made following the precast panel work at Mississippi River Lock 22 in precast panel technology and incorporated in the work at Troy Lock are documented previously in this report. The revisions greatly improved the efficiency of panel installation and increased productivity. However, some recommendations should be made for improvement of the precast panel work for future lock projects including:

a. Panel fabrication:

- (1) Panels should continue to be fabricated by an experienced and qualified precast manufacturer.
- (2) Panel anchors should be detailed to provide wider opening for greater tolerance in matching with drilled anchors and vertical reinforcing bars.
- (3) Eliminate 2-in. horizontal grouted joint between panels. Holes for shebolt anchors should be provided in lock panels (similar to river wall panels or small notches provided on top and bottom of panels).
- (4) Bottom of panel should be fabricated with a thin rubber bearing pad for improved leveling and bearing.
- (5) Taping of form joints by the precaster should not be permitted to eliminate taping marks on finished surfaces.
- (6) Mesh should be eliminated in river wall panels (5-1/2 in. thick).

b. Concrete removal and surface preparation:

- (1) Bottom shelf for setting panels should be full depth saw cut and any broken shelf areas repaired with concrete grout prior to placing panels. Use of recessed steel shims for leveling is recommended.
- (2) Wall surface in removal areas should be hydroblasted or sand blasted prior to panel erection, except that in freezing weather high pressure air should be substituted for hydroblasting to eliminate problems with ice buildup on the walls, scaffolds, etc.

c. Installation of panels:

- (1) Alternate contractor designed forming supports (i.e. strong backs) should be reviewed by the panel designer prior to use.

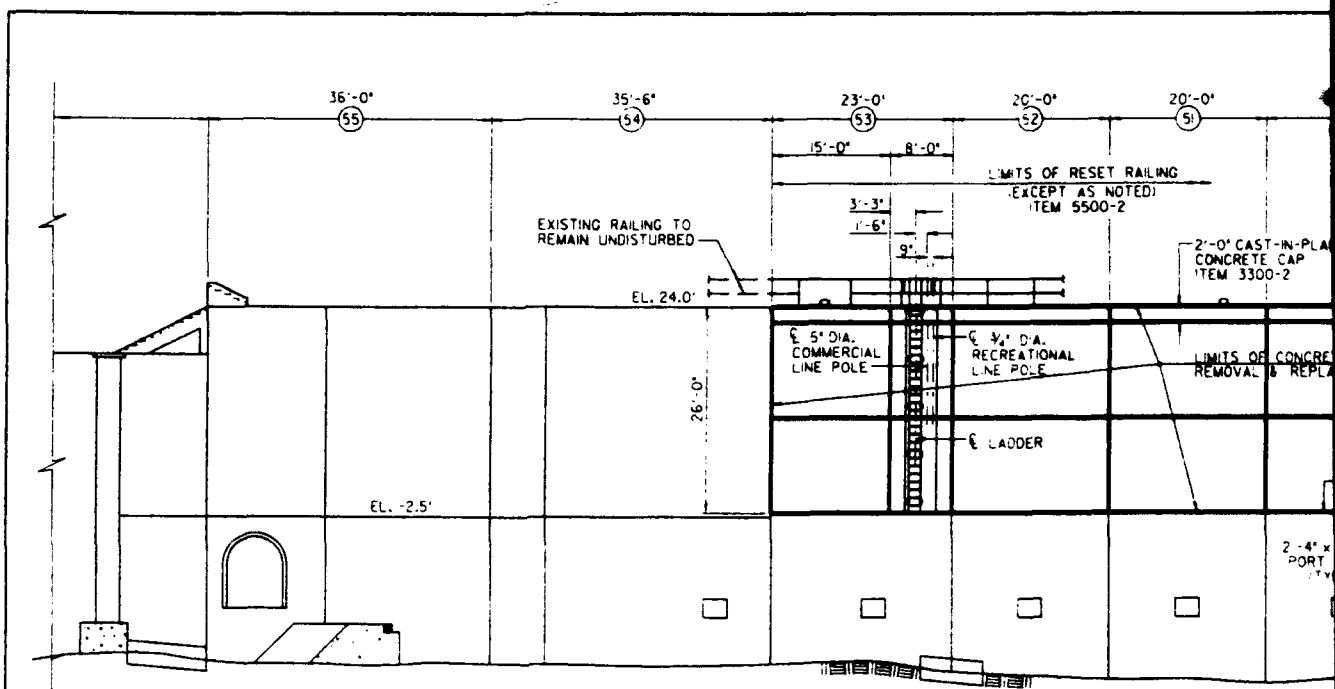
- (2) Specifications should provide additional information concerning infill placement operations including required time interval between successive placements.
- d. *Joint details:* Horizontal joints (2-in.) should be eliminated to reduce joint finishing work and leakage.
- e. *Control of cracking:*
 - (1) Surface sealing of fine cracks (< 0.006-in.) should be eliminated since these cracks are not detrimental to quality.
 - (2) Suggestions noted in Item c also apply.
- f. *Use of panels:* Precast panels should be considered for the repair of other vertical concrete surfaces such as approach walls, valve pit walls, and pier faces, especially near the waterline where expensive coffer-daming can be avoided.
- g. *Special panels:*
 - (1) Continued use of special panels for ladders, line poles, etc. where multiple pieces are required. In unique areas not subject to ship abrasion, cast-in-place repair may be more efficient.
 - (2) Ladders and line poles should not be combined at the same location due to the problems of ship impact on the wider wall opening. Also, line poles and ladders should be recessed deeper from the wall surface and side bars increased in thickness to prevent damage from vessels.

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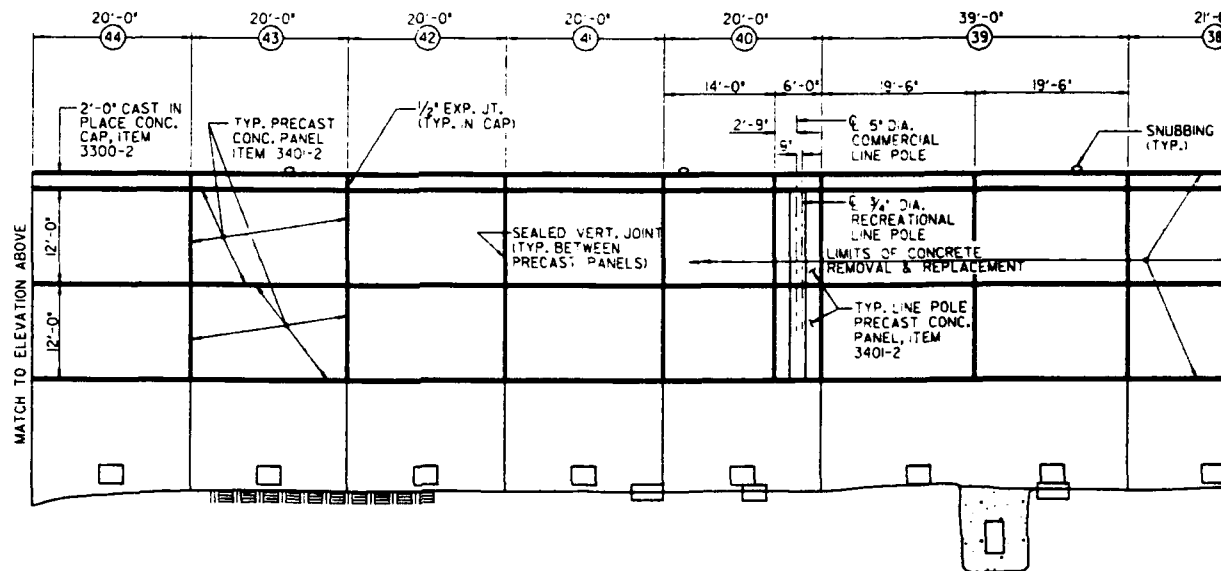
Pace, C. E., Campbell, R., and Wong, S. (1981). "Engineering condition survey and evaluation of Troy Lock and Dam, Hudson River, New York, Evaluation and Rehabilitation," Miscellaneous Paper C-78-6, Report 2, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

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WEST LOCK WALL - ELEVATION

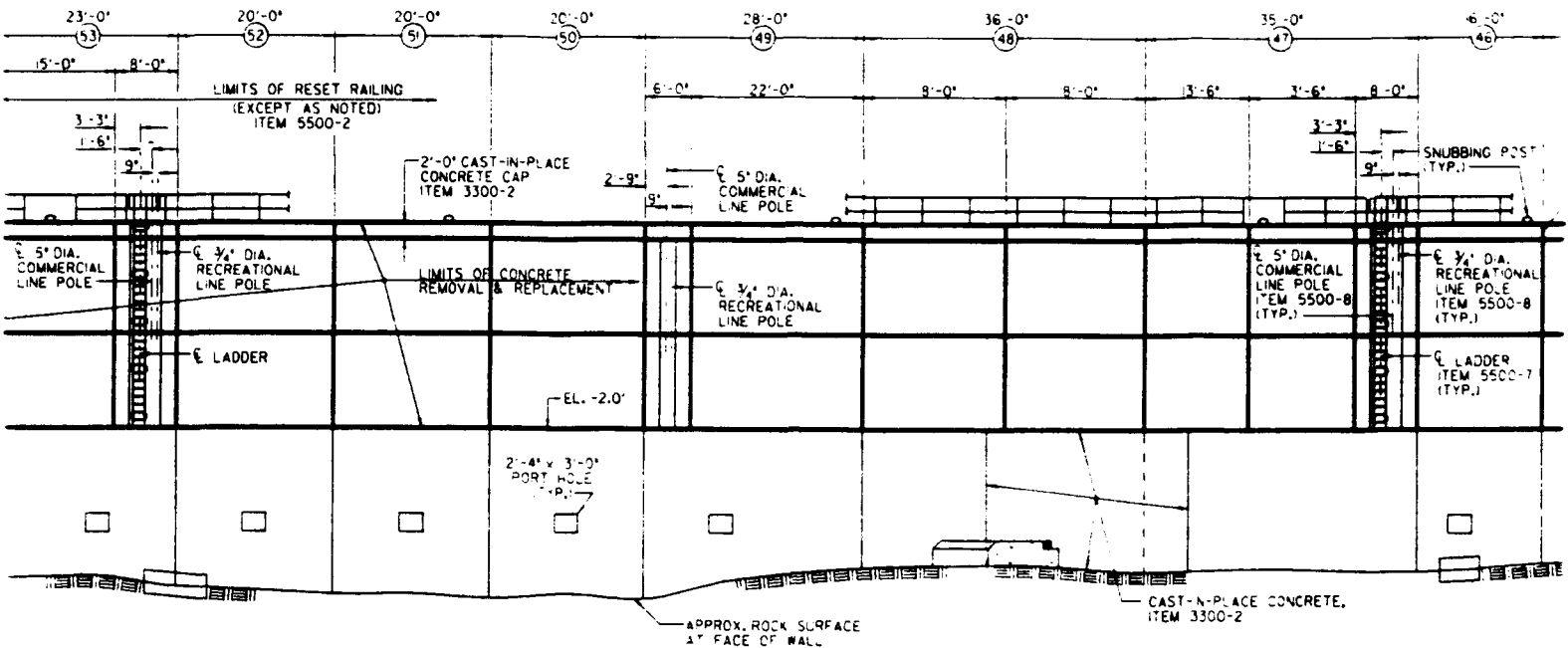
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WEST LOCK WALL - ELEVATION

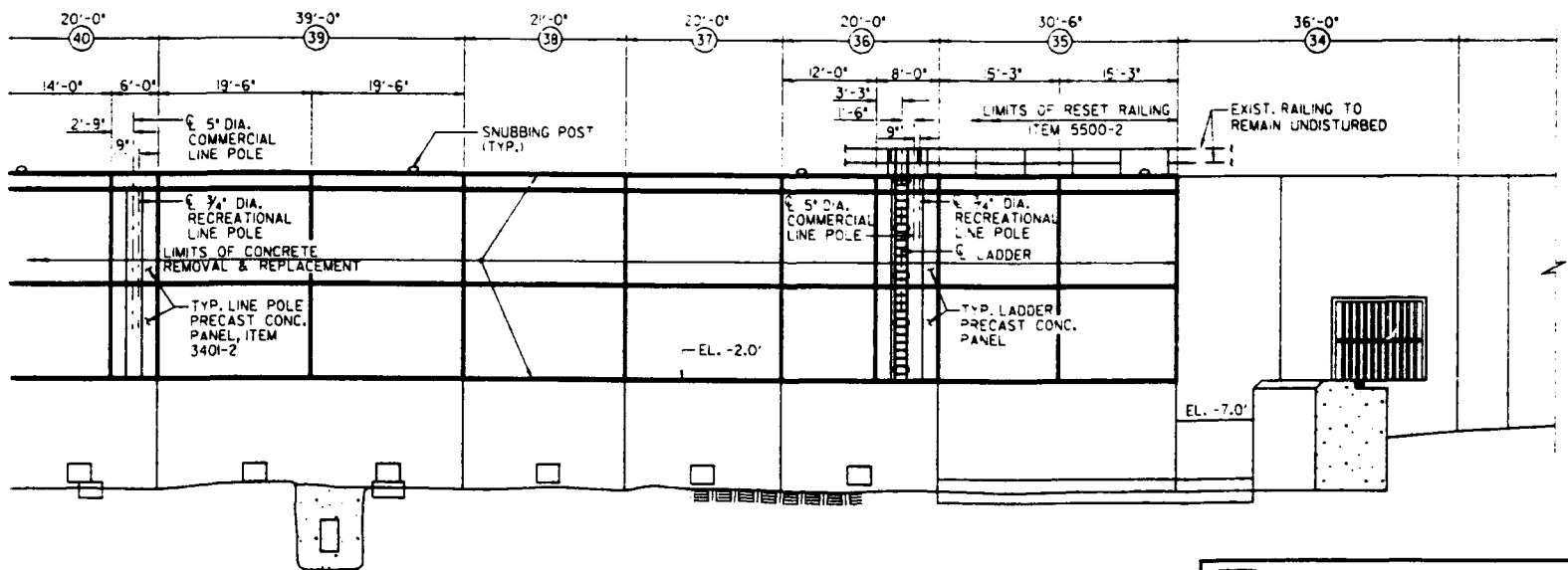
SCALE: 1" = 20'-0"

NOTE: EAST LOCK WALL ELEVATION IS SIMILAR



WEST LOCK WALL - ELEVATION

SCALE: 1" = 20'-0"



WEST LOCK WALL - ELEVATION

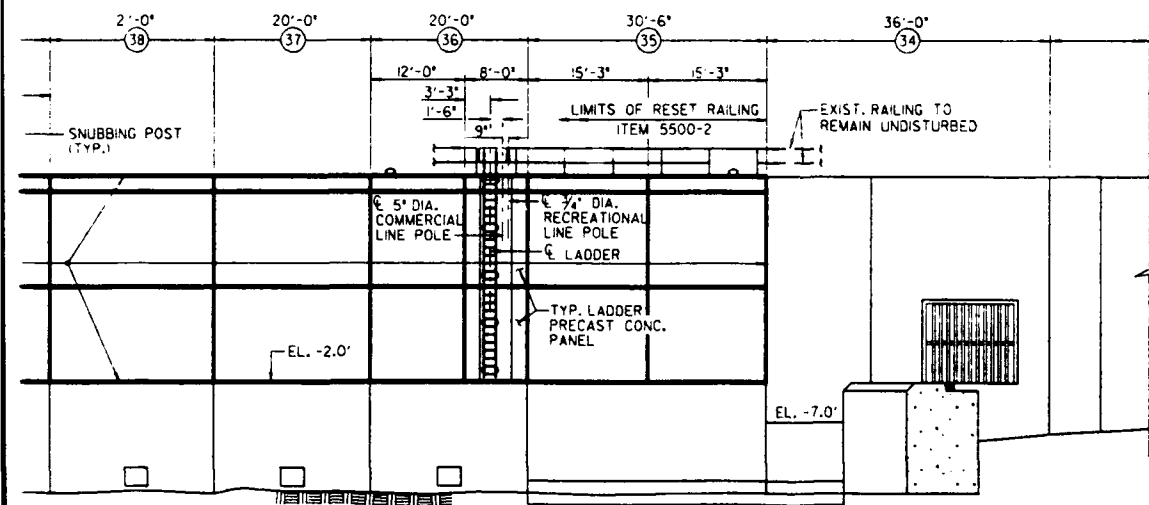
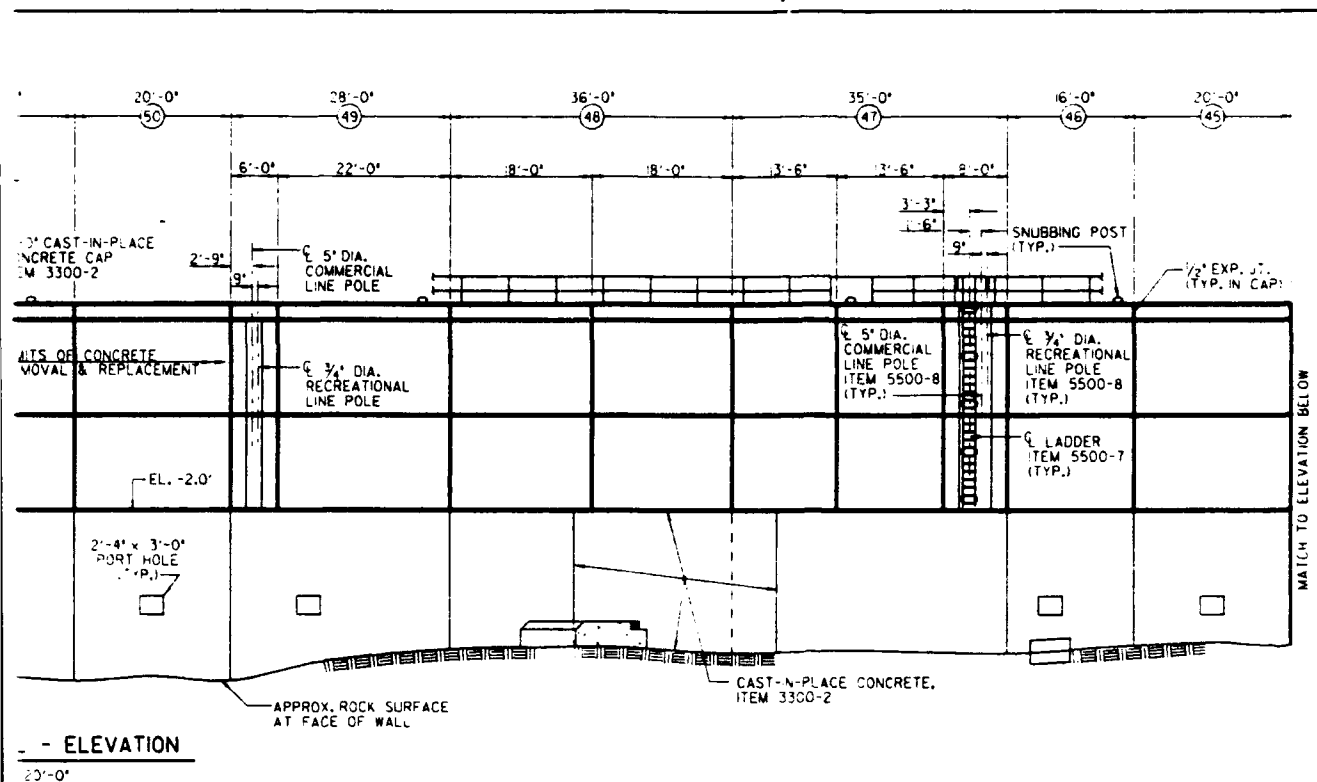
SCALE: 1" = 20'-0"

TROY LOCK AND D
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CONCRETE REPAIR PROJECT

WEST LOCK WALL - ELEVATION

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ROCHESTER, N.Y. 14614

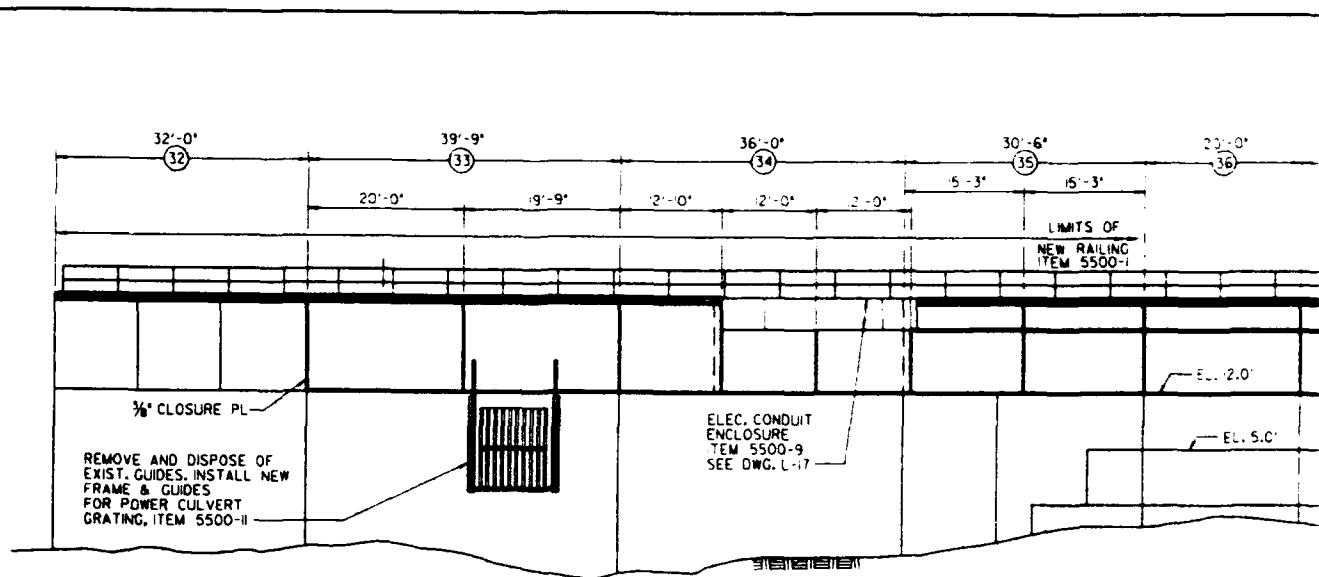


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WEST LOCK WALL - ELEVATION

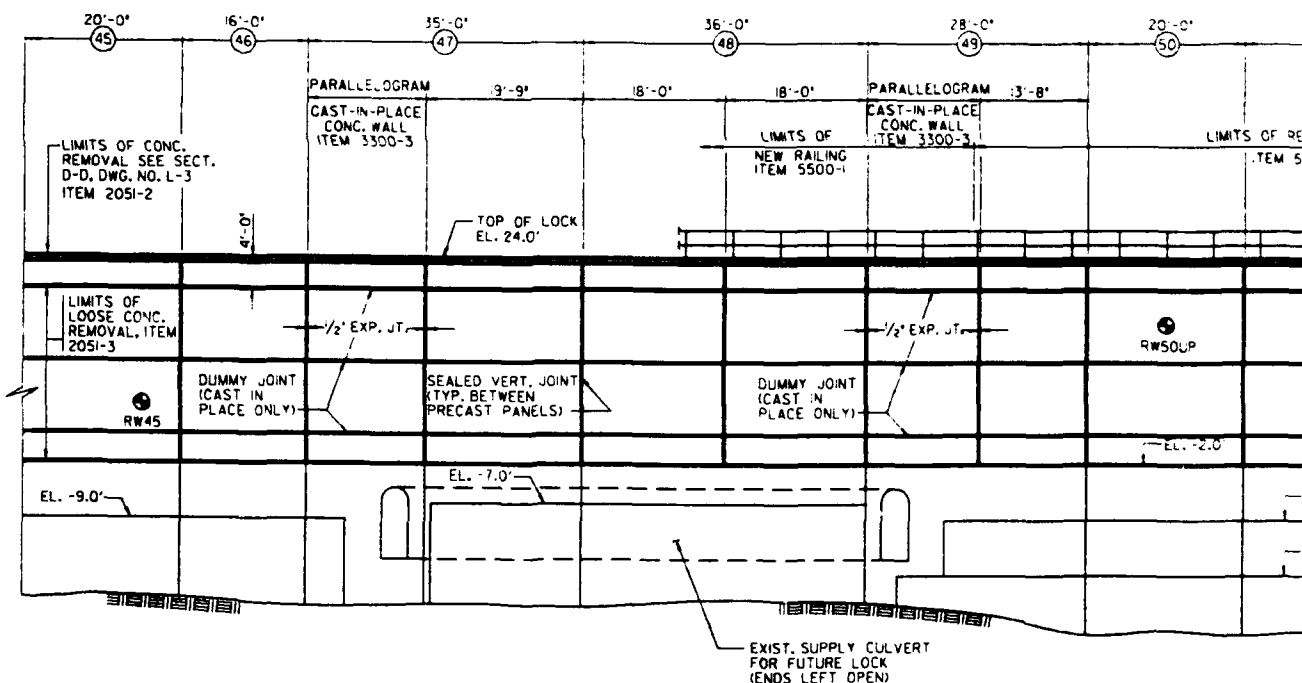
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Date: **SEPT. 1992**
Plate: **1**



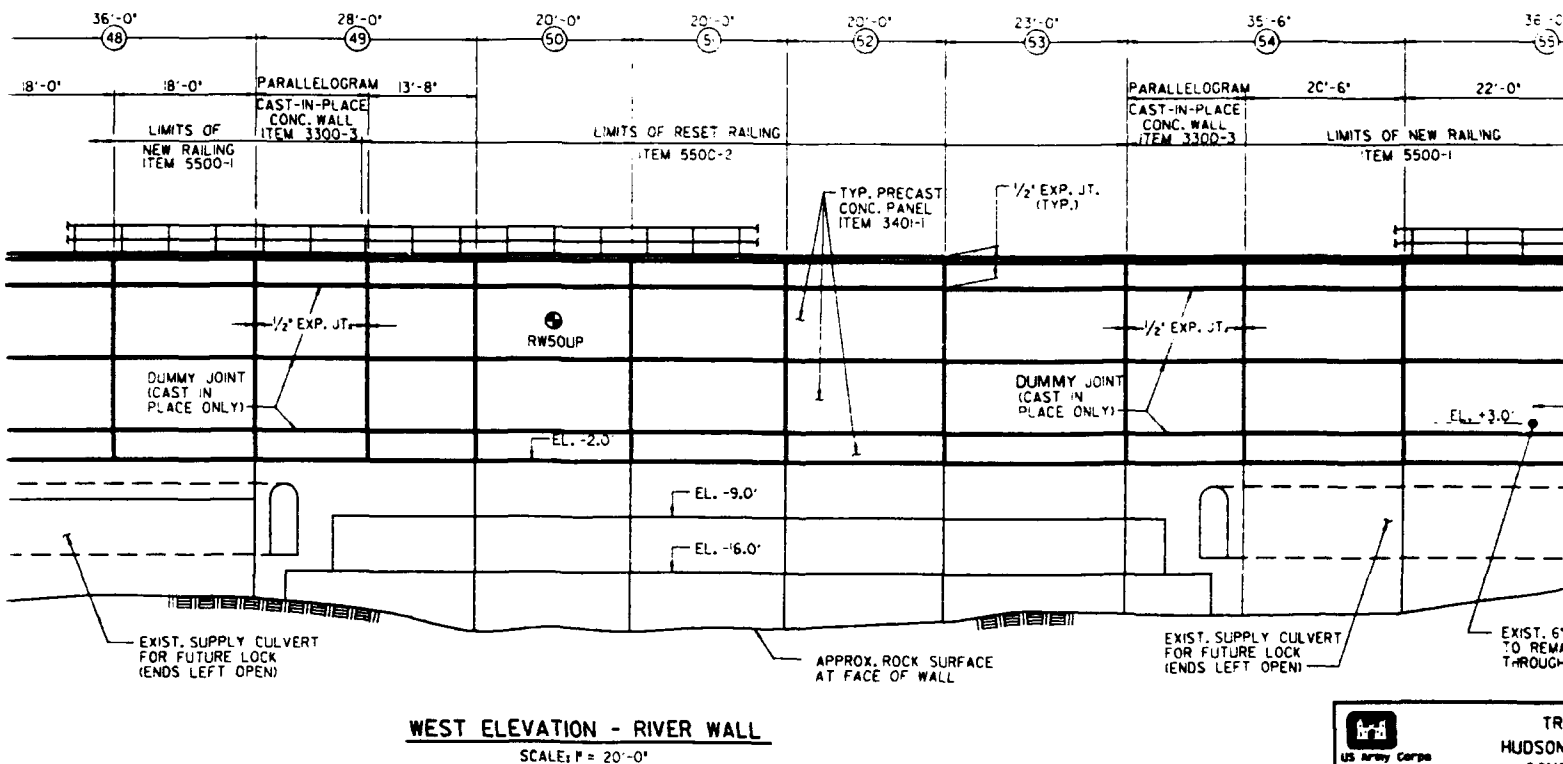
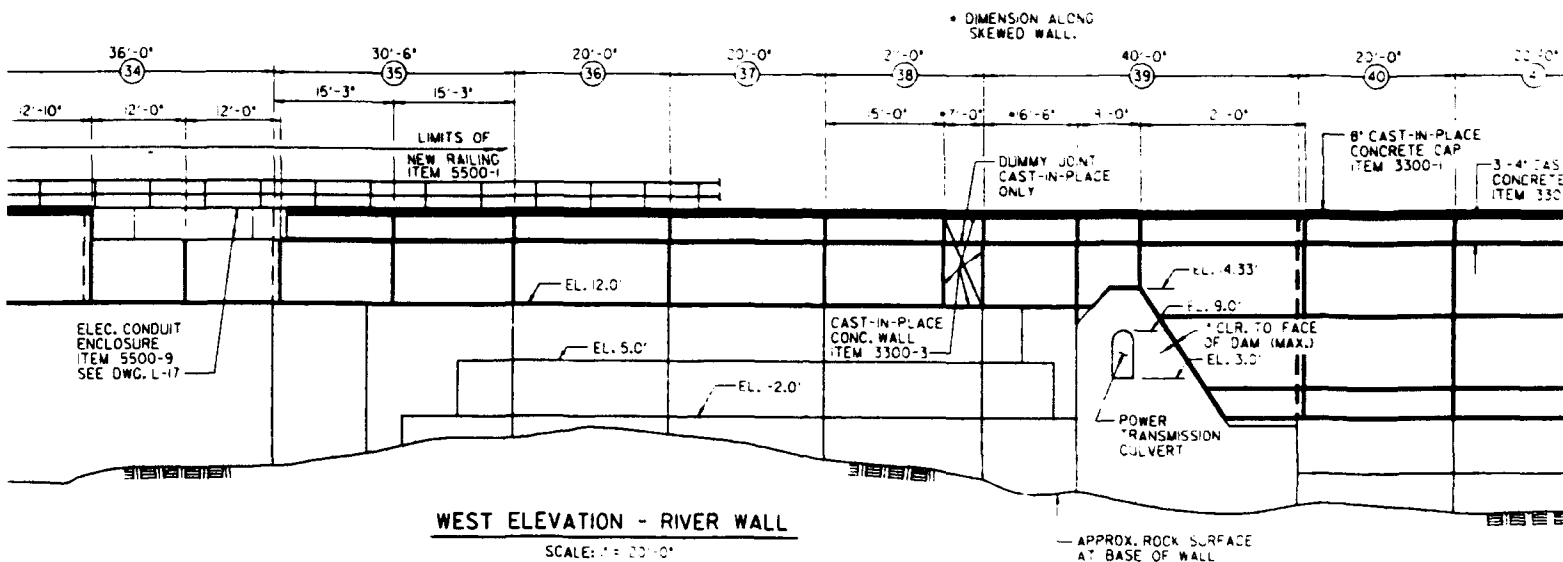
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WEST ELEVATION - RIVER

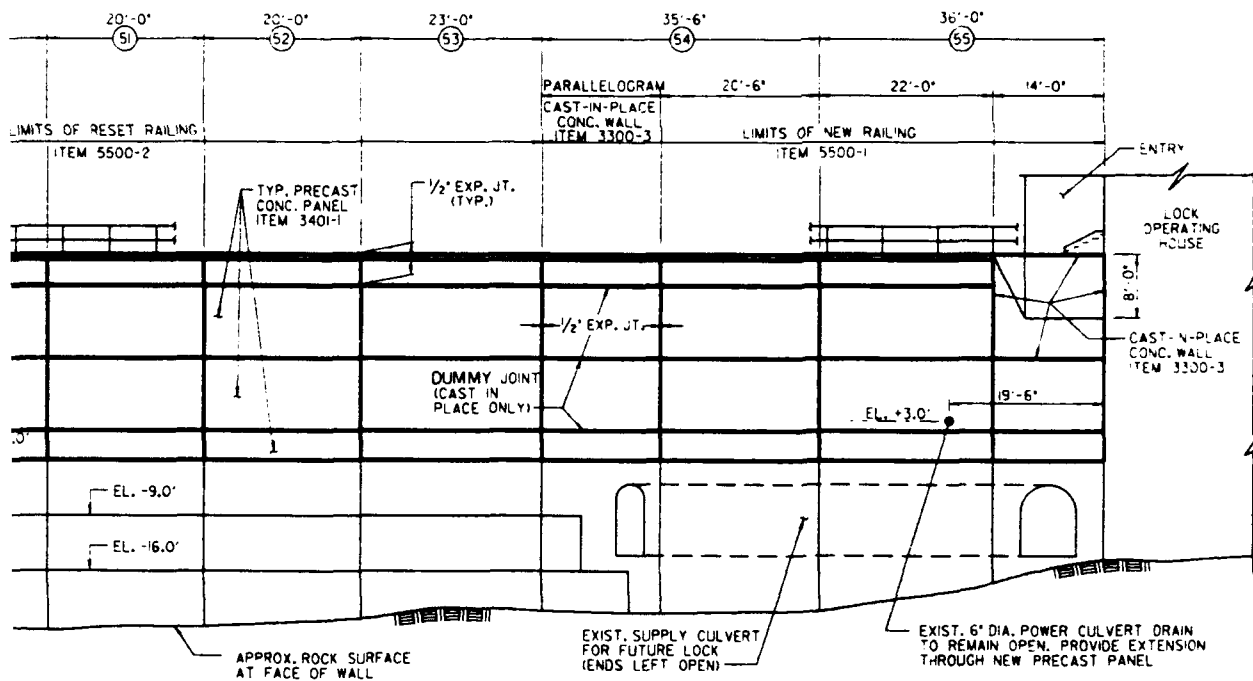
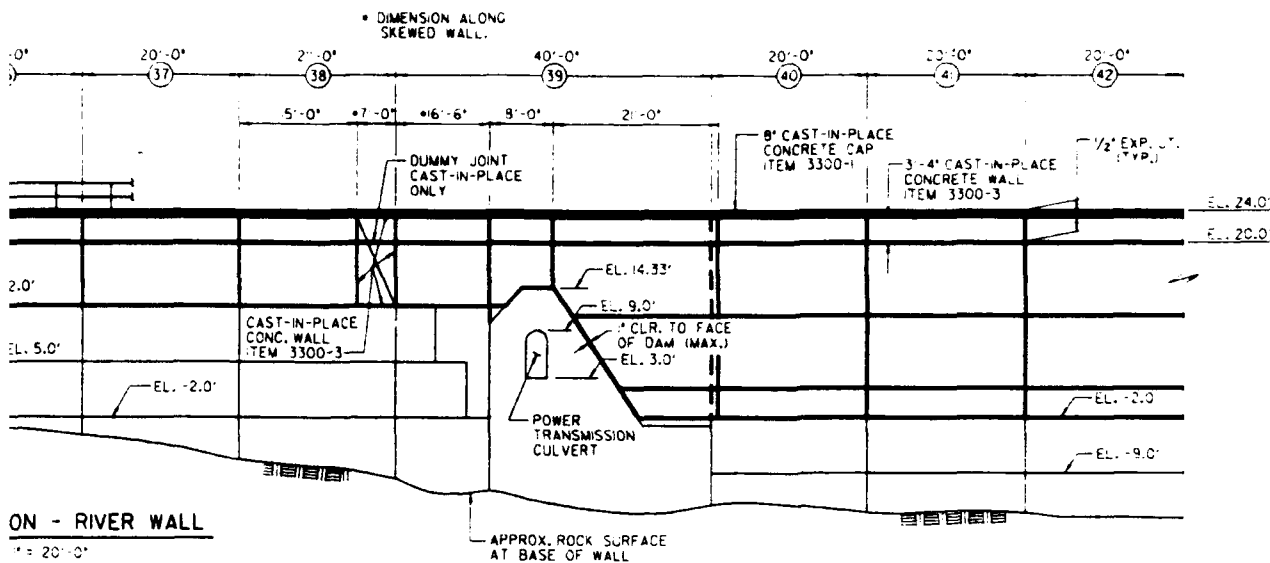
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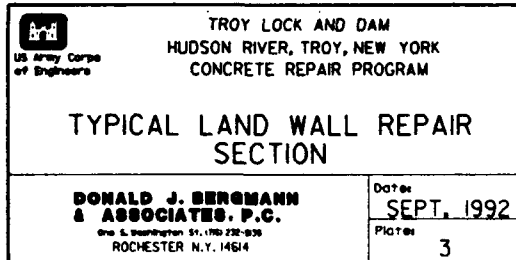


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WEST ELEVATION - RIVER WALL

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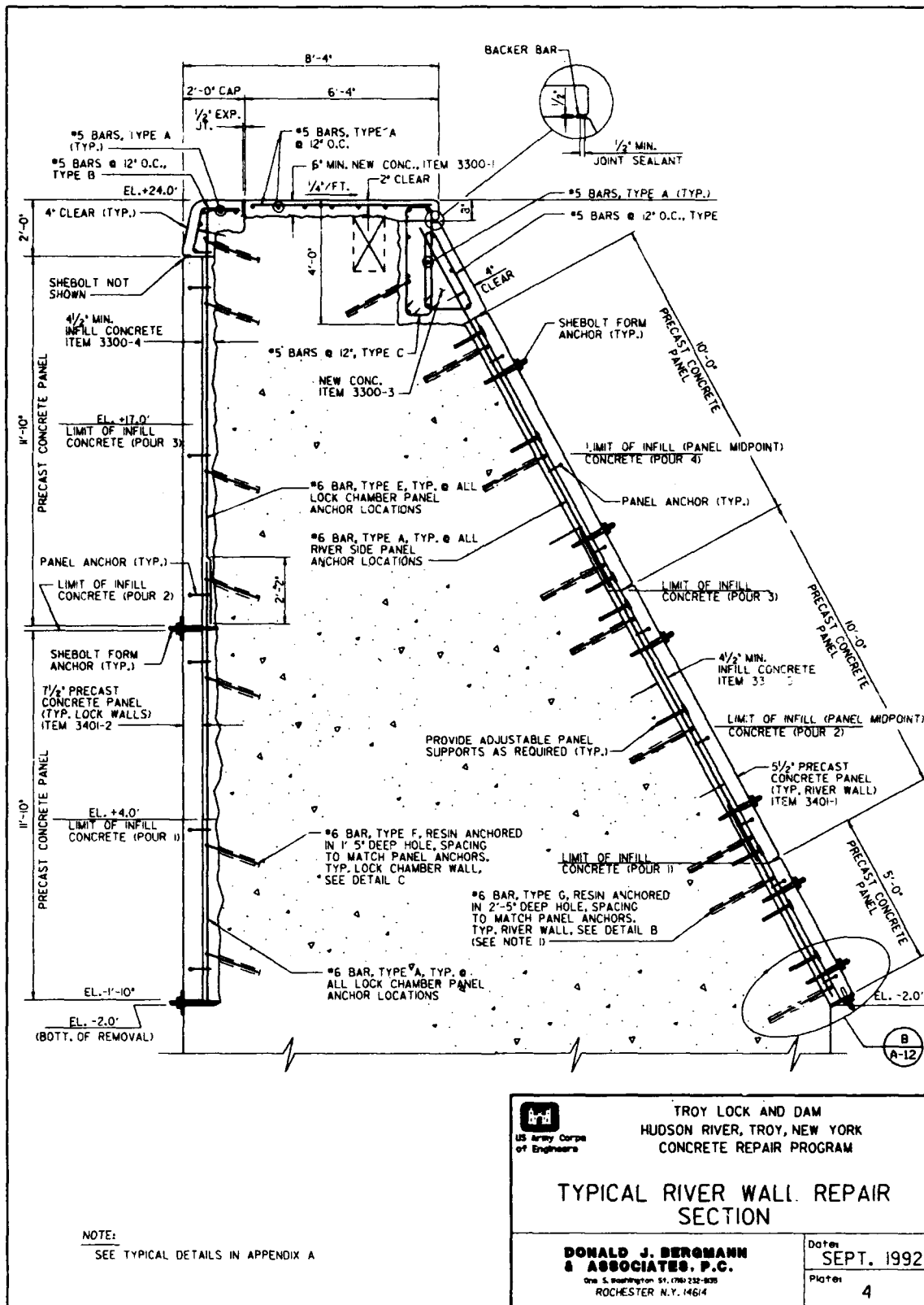
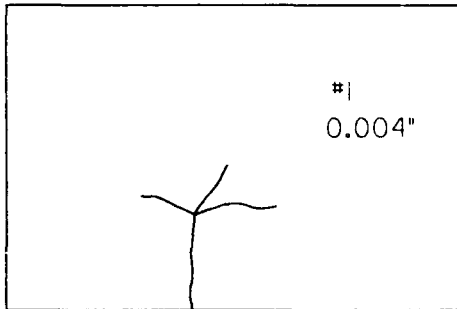
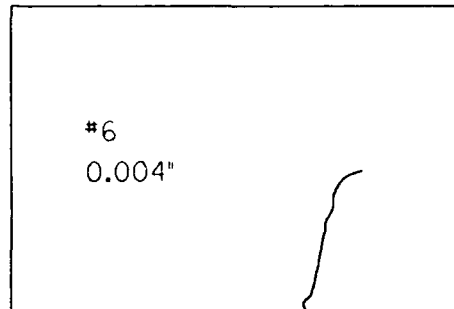


Plate 4

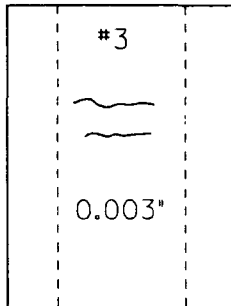
PANEL 7s10s - MONO 4



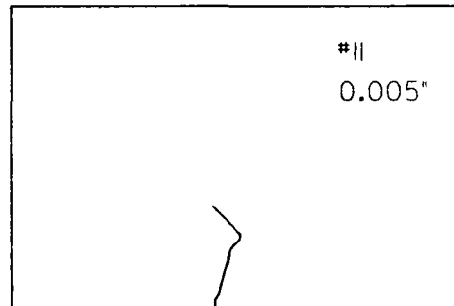
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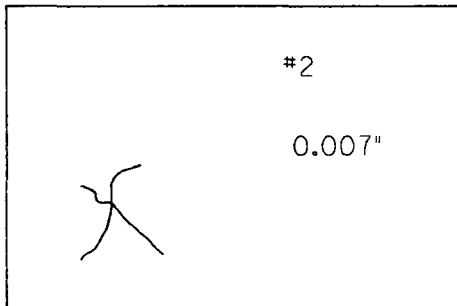
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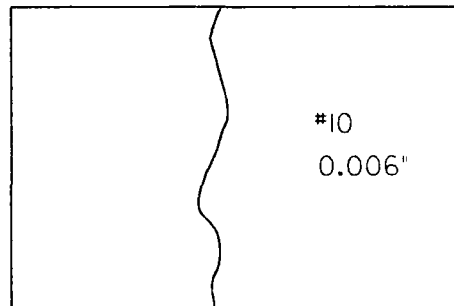
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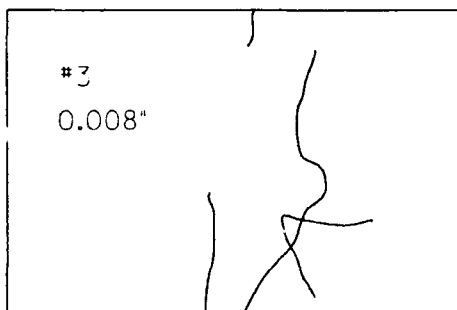
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PANEL 7s17 - MONO 21



PANEL 7s21 - MONO 9



NOTES:

1. Number noted is order of crack as located in the field.
2. Dimension noted is maximum crack width measured.



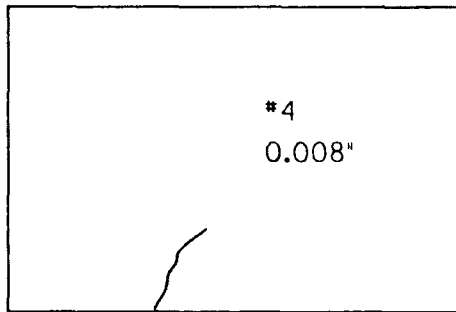
TROY LOCK AND DAM
HUDSON RIVER, TROY, NEW YORK
CONCRETE REPAIR PROGRAM

**LOCK CHAMBER PRECAST PANELS
EAST WALL CRACKS**

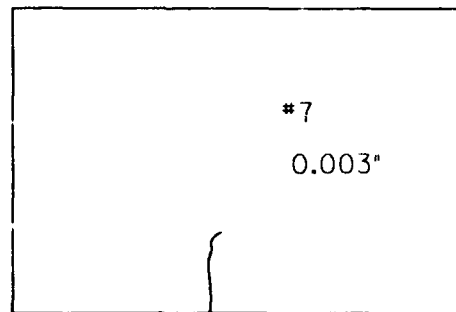
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Plate: 5

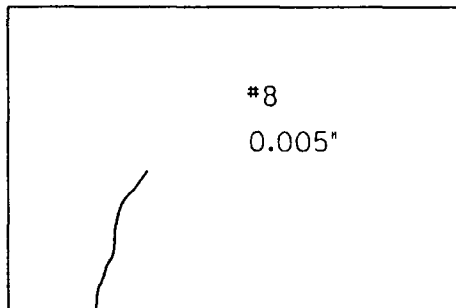
PANEL 7s28 - 38



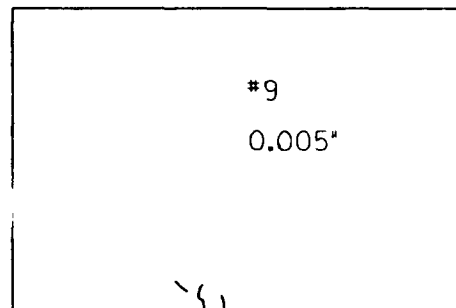
PANEL 7s21 - MONO 42



PANEL 7s14 (North) - MONO 39



PANEL 7s26 - MONO 45



NOTES:

1. Number noted is order of crack as located in the field.
2. Dimension noted is maximum crack width measured.



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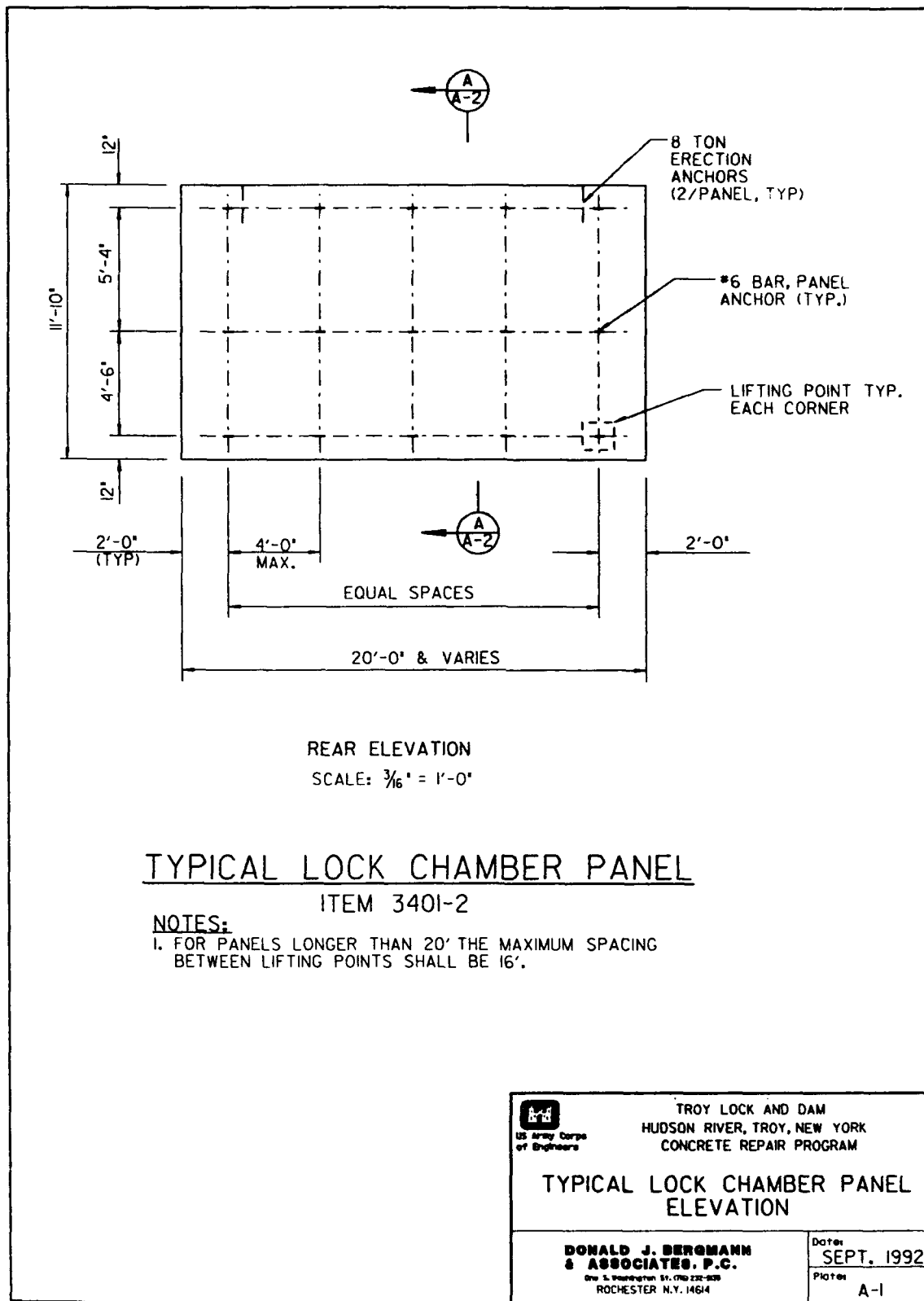
**LOCK CHAMBER PRECAST PANELS
WEST WALL CRACKS**

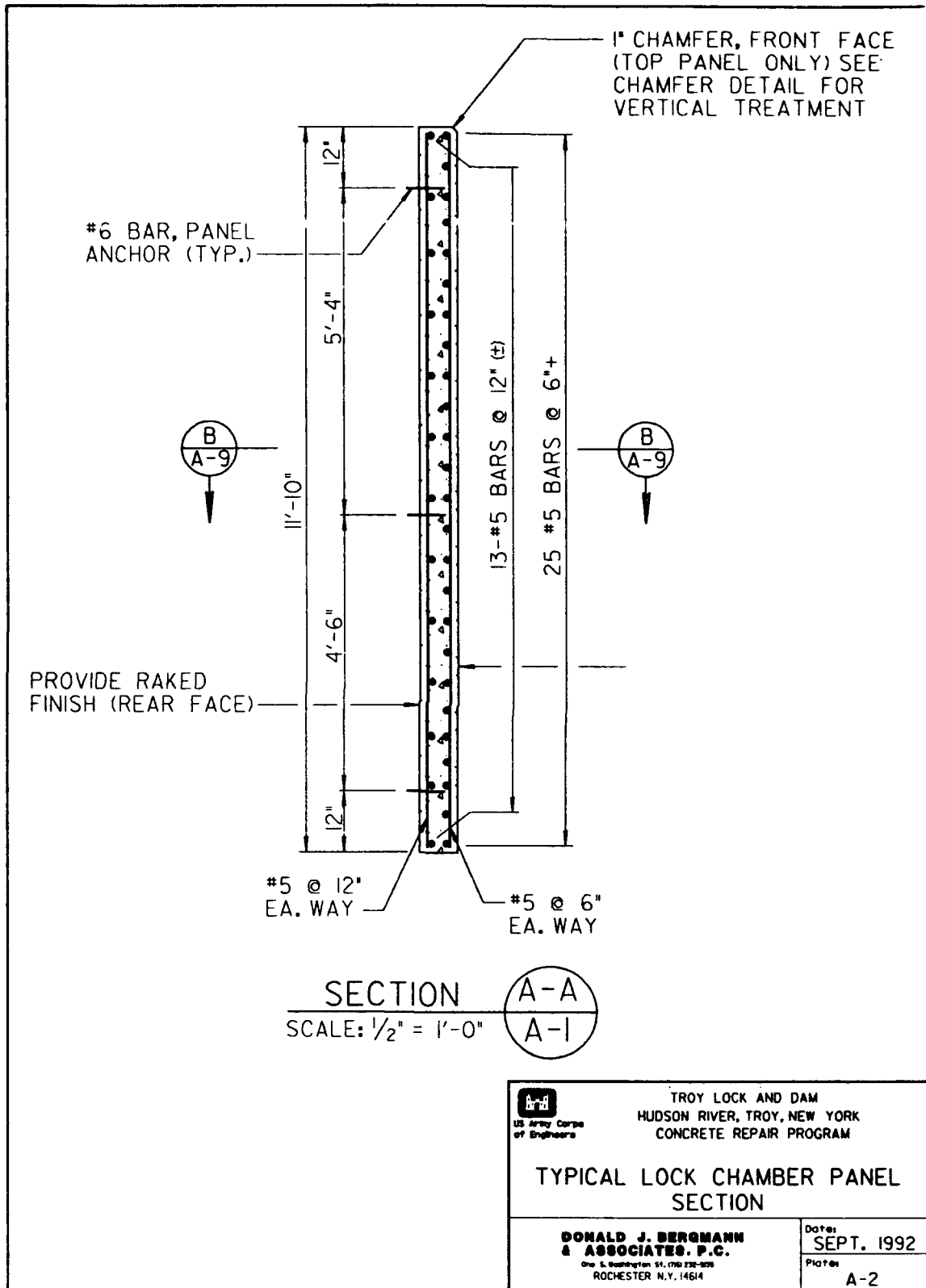
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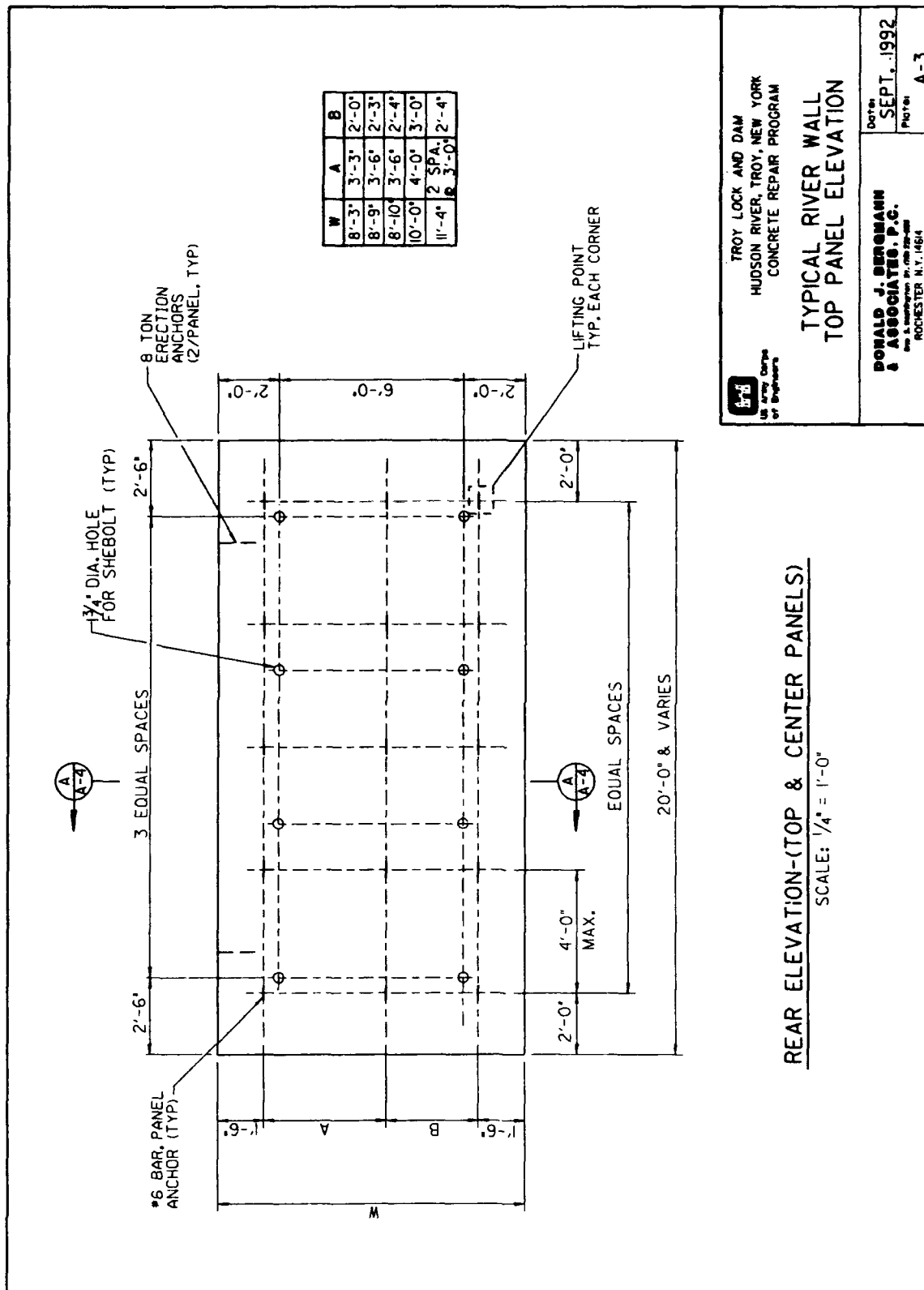
Date: **SEPT. 1992**
Plate: **6**

Appendix A

Panel Details







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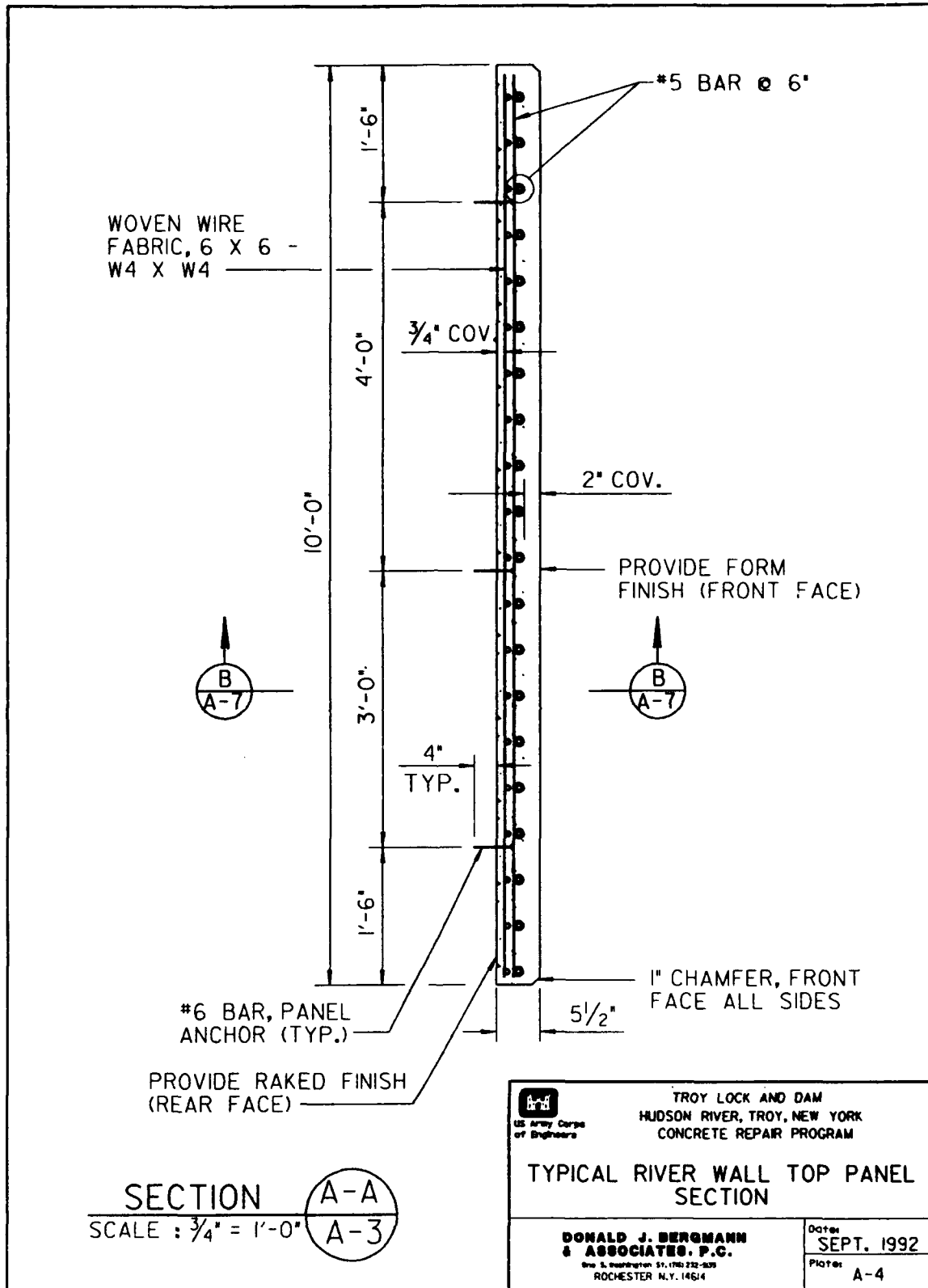
TYPICAL RIVER WALL TOP PANEL ELEVATION

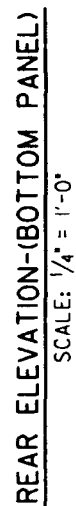
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DATE: SEPT., 1992
PROJECT: A-3

REAR ELEVATION-(TOP & CENTER PANELS)

SCALE: 1/4" = 1'-0"



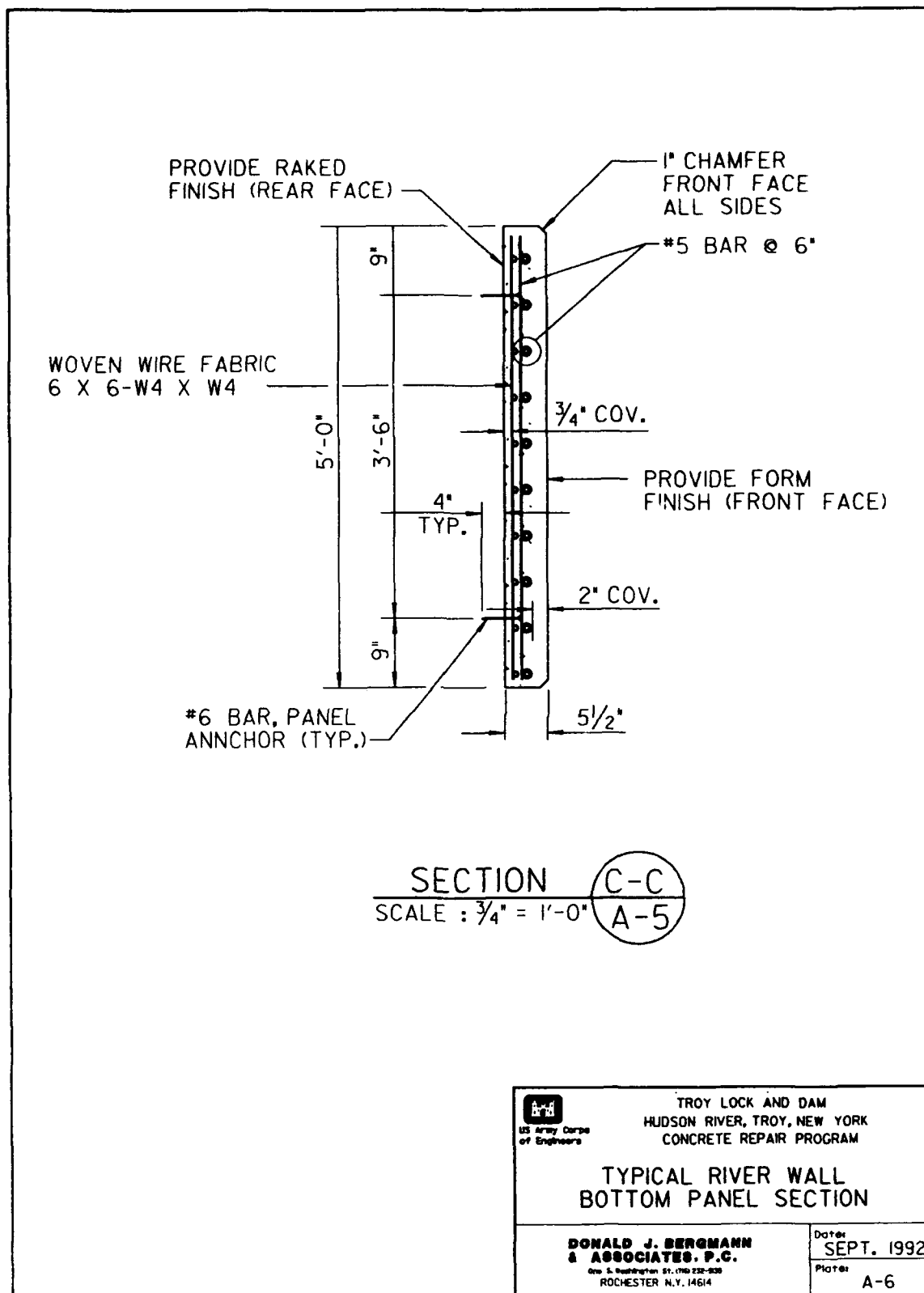


SCALE: $\frac{1}{4}'' = 1'-0''$

TYPICAL RIVER WALL
BOTTOM PANEL ELEVATION

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DATE: SEPT. 1992
PLACE: A-5



WOVEN WIRE FABRIC
6 X 6-W4 X W4

$\frac{3}{4}$ " COV.

#6 BAR, PANEL
ANCHOR (TYP.)

2" COVER
(TYP.)

8"
12"
(TYP.)

4"

#5 BARS @ 6"

5 1/2"

SECTION

B-B

SCALE: 1"=1'-0"

A-4

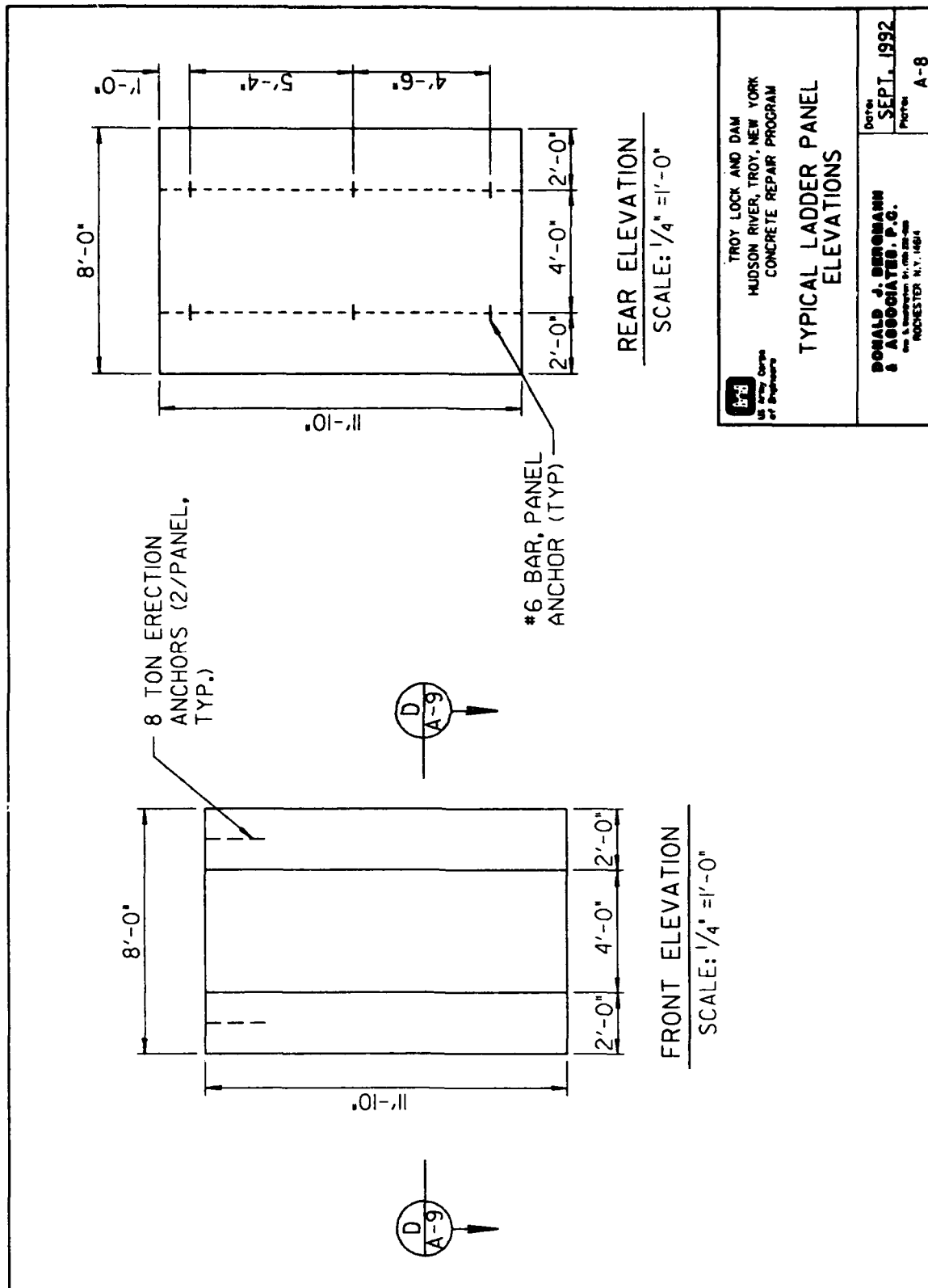


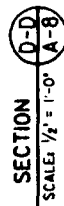
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TYPICAL RIVER WALL PANEL
SECTION

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ROCHESTER N.Y. 14614

Date: **SEPT. 1992**
Plot of:
A-7





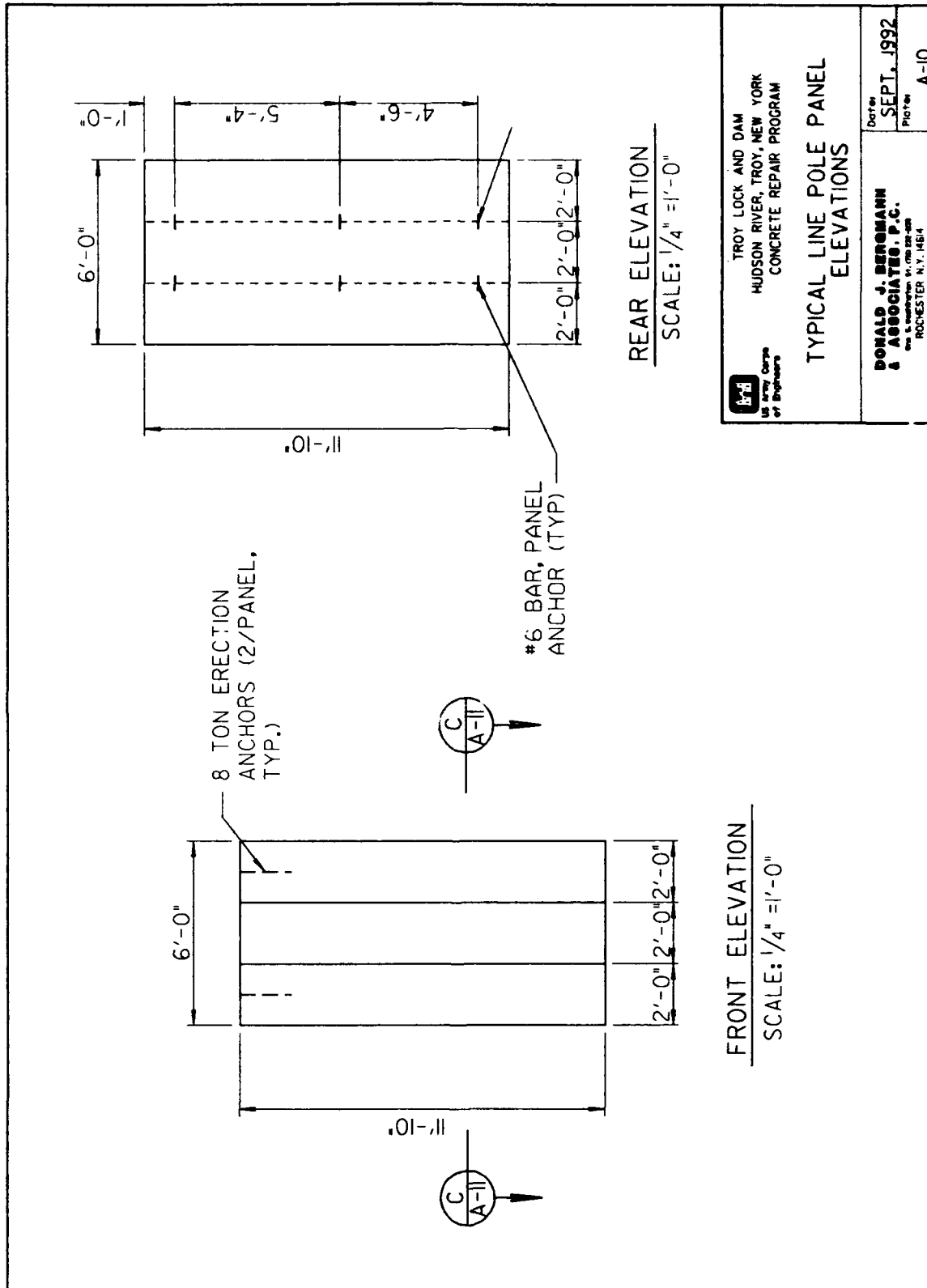
**TROY LOCK AND DAM
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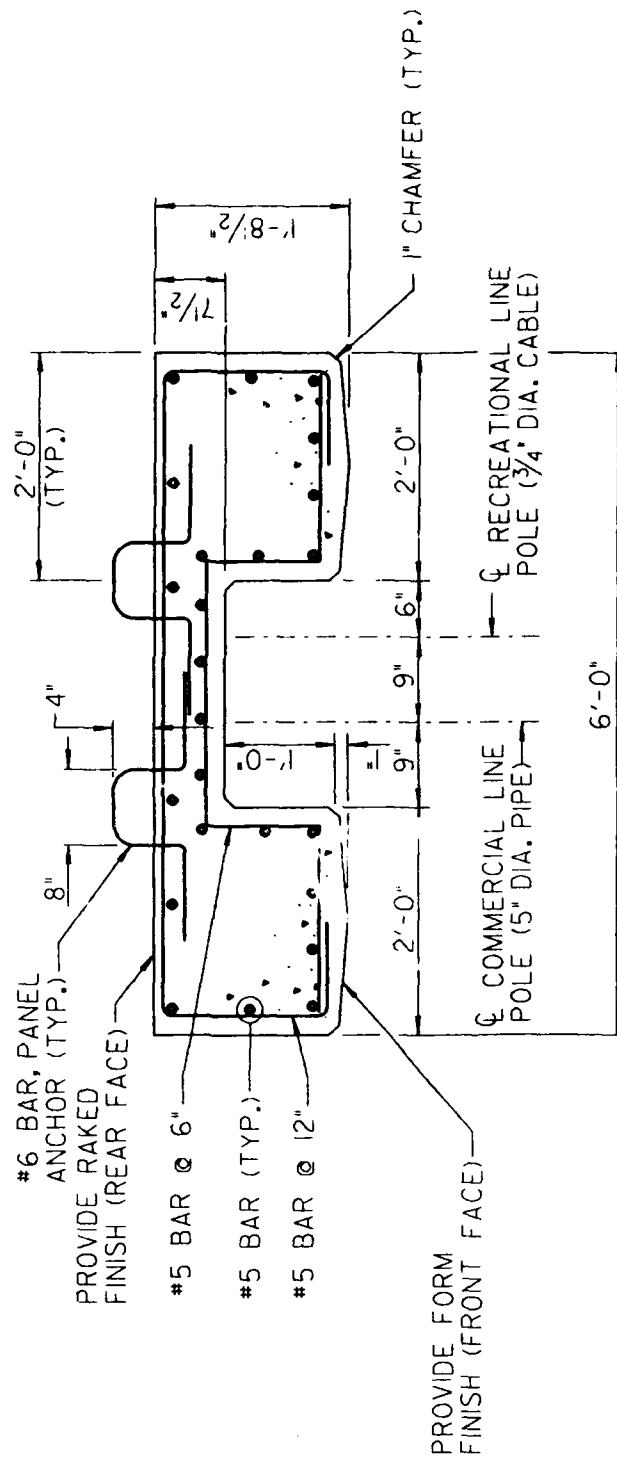
TYPICAL LADDER PANEL AND LOCK PANEL SECTIONS

**DONALD J. BERGMANN
& ASSOCIATES, P.C.**
ONE S. BRADSTREET ST., (100) 222-0000
ROCHESTER, N.Y. 14614

Date: SEPT. 1992

Plate: A-9





SECTION $\frac{3}{4}'' = 1'-0''$

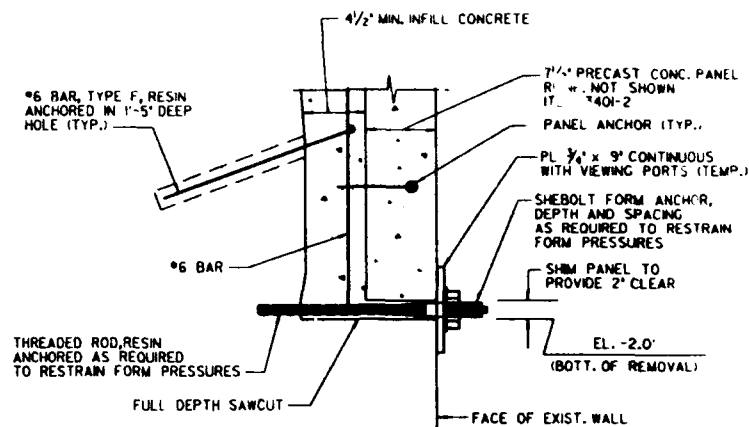
C-C
A-10

ES&S
Engineering & Surveying
TROY LOCK AND DAM
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CONCRETE REPAIR PROGRAM

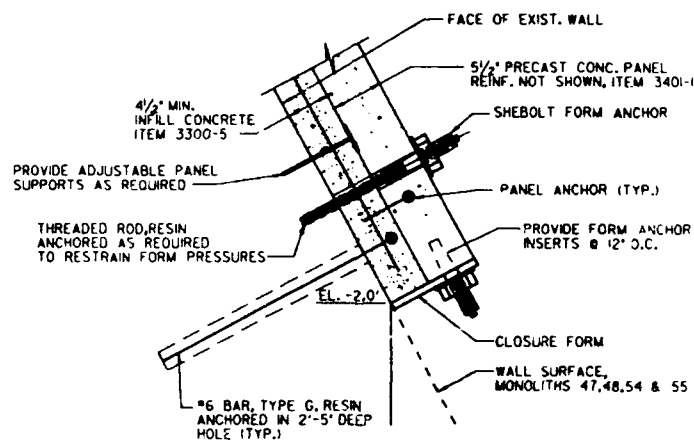
TYPICAL LINE POLE PANEL SECTION

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Civil Engineers
ROCHESTER, N.Y. 14614


Date: **SEPT. 1992**
Project: **A-11**

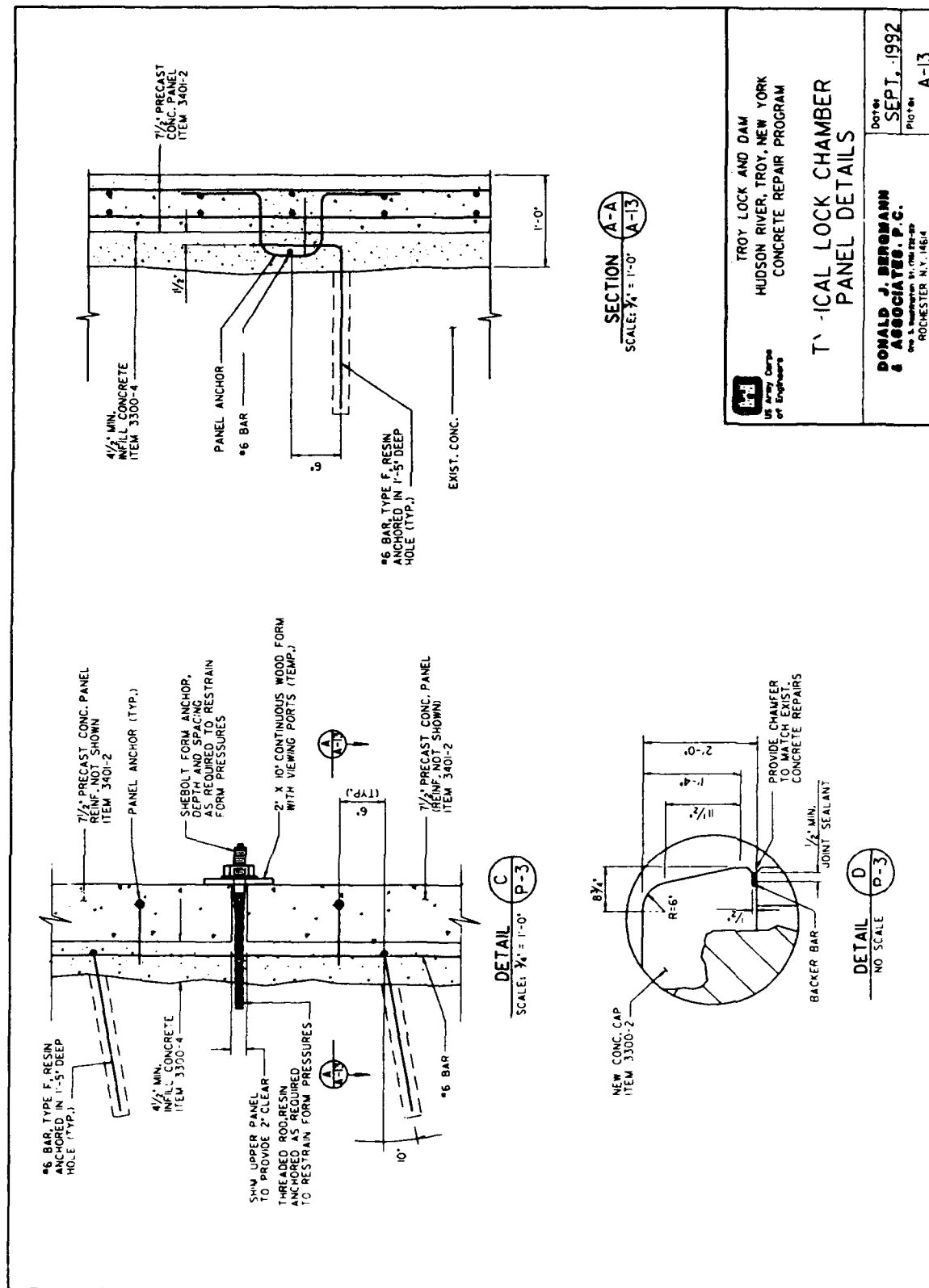


DETAIL A
SCALE: 3/4" = 1'-0"
(LOCK CHAMBER)



DETAIL B
SCALE: 3/4" = 1'-0"
(RIVER WALL)

 US Army Corps of Engineers	TROY LOCK AND DAM HUDSON RIVER, TROY, NEW YORK CONCRETE REPAIR PROGRAM	
	TYPICAL PANEL DETAILS	
DONALD J. BERGMANN & ASSOCIATES, P.C. One E. Washington St., 17th Floor Rochester, N.Y. 14614	Date: SEPT. 1992	Plate: A-12



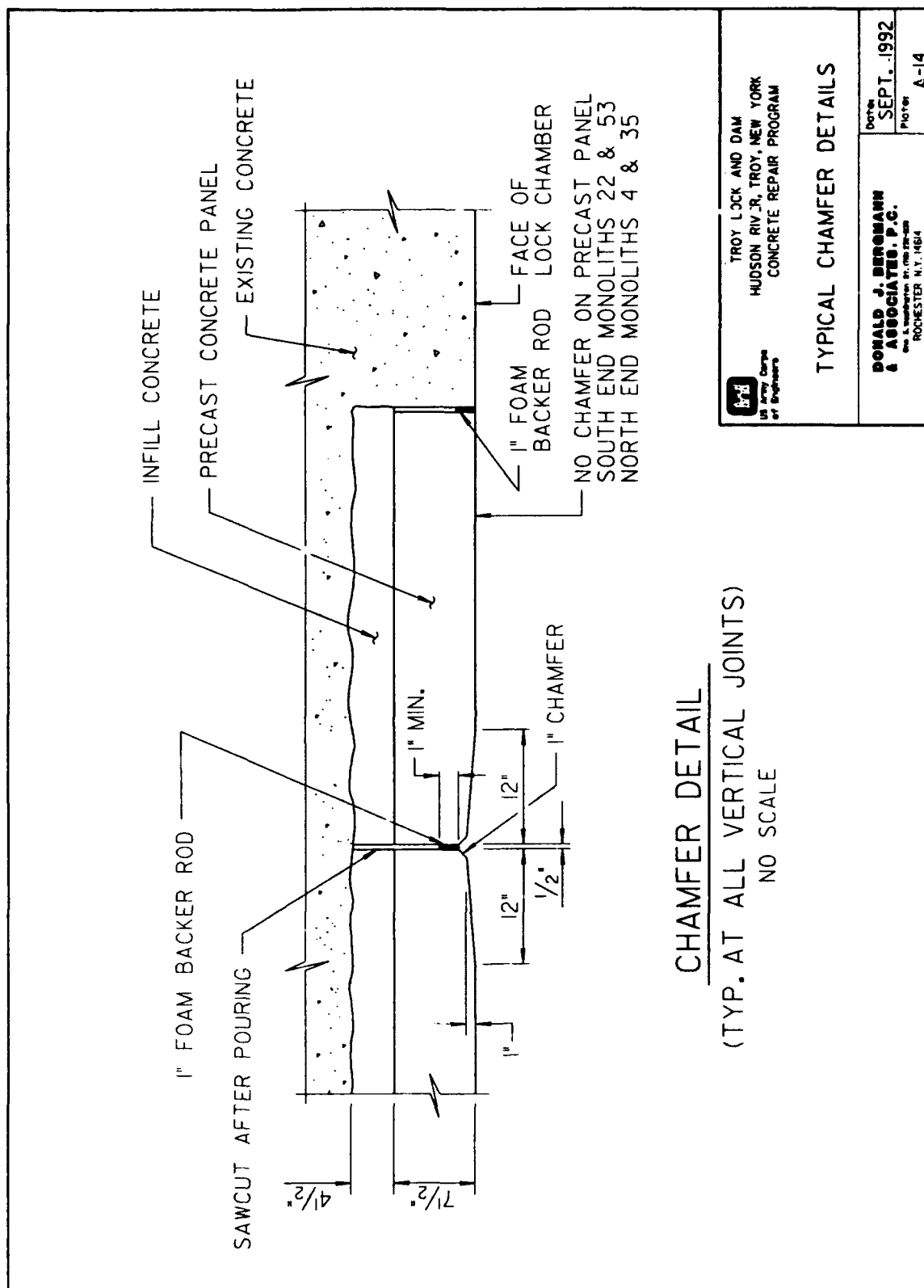
US Army Corps of Engineers

TROY LOCK AND DAM
HUDSON RIVER, TROY, NEW YORK
CONCRETE REPAIR PROGRAM

TYPICAL LOCK CHAMBER PANEL DETAILS

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ROCHESTER, N.Y. 14614

DATE: **SEPT. 1992**
PROJECT: **A-13**



Appendix B

Contract Specifications

SECTION 03401

PRECAST CONCRETE

1. SCOPE. The work covered by this section consists of furnishing all equipment, labor, and materials for providing and installing structural precast concrete panels for lock chamber and river wall surface repairs as indicated on the drawings. Precast units are all non-prestressed.

2. RELATED WORK SPECIFIED ELSEWHERE.

2.1 FORMWORK FOR CONCRETE. Section 03100

2.2 STEEL BARS AND ACCESSORIES FOR CONCRETE REINFORCEMENT.
Section 03200

2.3 EXPANSION, CONTROL, AND CONSTRUCTION JOINTS IN CONCRETE.
Section 03250

2.4 CAST-IN-PLACE STRUCTURAL CONCRETE. Section 03300

3. REFERENCE STANDARDS. The following publications of the issues listed below, but referred to thereafter by the basic designation only form a part of this specification to the extent indicated by the references thereto.

3.1 American Concrete Institute (ACI) Standards.

ACI 214-77 (R 1989) Recommended Practice for Evaluation of
Strength Test Results of Concrete

ACI 318-89 Building Code Requirements for Reinforced
Concrete

3.2 American Society for Testing and Materials (ASTM) Standards.

C 31-85 (CRD-C 11) Making and Curing Concrete Test Specimens
in the Field

C 33-86 (CRD-C 133) Concrete Aggregates

C 39-86 (CRD-C 14) Compressive Strength of Cylindrical
Concrete Specimens

C 150-85a (CRD-C 201) Portland Cement

C 172-82 (CRD-C 4) Sampling Freshly Mixed Concrete

C 231-82 (CRD-C 41) Air Content of Freshly Mixed Concrete by the
Pressure Method

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C 260-86 (CRD-C 13)	Air Entraining Admixtures for Concrete
C 494 -86 (CRD-C 87)	Chemical Admixtures for Concrete
C 617-85b (CRD-C 29)	Capping Cylindrical Concrete Specimens

3.3 NYSDOT Standard Specifications for Construction and Materials

3.4 Prestressed Concrete Institute (PCI)

MNL 116-77	Manual for Quality Control for Plants and Production of Precast Prestressed Concrete Products
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4. QUALITY ASSURANCE.

4.1 Plant and Specialist Qualifications. Precast concrete shall be the product of a manufacturer having five years minimum experience in producing similar products and with capacity and facilities for producing materials of the quality and finish specified herein, and of the quantity indicated, without delay to progress of work. The manufacturer shall have an established quality control program in effective operation at this plant, attested to by a current certification of the plant, by PCI or NPCA. The manufacturer must have an automated batch plant approved by the NYSDOT.

4.1 Test Panels. One test panel for the lock walls and one test panel for the river walls with minimum dimensions of 36" x 36" x design wall thickness shall be cast and approved by the Contracting Officer prior to any casting of full size wall panels. Panels shall include edge chamfer, panel anchor and shebolt hole (river panel only) to simulate actual panel.

4.2 Sampling and Testing.

4.2.1 General: Samples and tests required below shall be made at the Contractor's expense. The tests shall be performed by an approved independent commercial testing laboratory or at a PCI or NPCA Certified Plant. Submit certified concrete test reports including all test data and results.

4.2.2 Concrete. Concrete shall be sampled and cylinders made in strict accordance with ASTM C 172 and ASTM C 31.

4.2.2.1 Concrete Test Cylinders. At least four test cylinders per casting bed shall be made each day the bed is used. A minimum of two test cylinders per day or 50 cubic yards of concrete or fraction thereof, whichever results in the most cylinders, shall be made to verify the attainment of the specified strength.

4.2.2.2 Cylinder Making. Cylinders shall be made as near as possible to the location where they will be cured and shall not be disturbed in any way from one half hour after casting until they are either 20 ± 4 hours old or ready to be tested. Concrete in

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after casting until they are either 20 ± 4 hours old or ready to be tested. Concrete in cylinder may be consolidated by rodding or by vibration as specified in ASTM C 31. If vibrators are used, techniques shall be used which will preclude segregation.

4.2.2.3 Cylinder Curing.

4.2.2.3.1 Test cylinders shall be cured with and by the same methods as the members they represent. In lieu of actual curing with the members, cylinders may be cured in curing chambers correlated in temperature and humidity with the beds. In such a case, the correlation shall be constantly verified by use of recording thermometers in the curing chambers and comparison with the temperature records of beds, and by use of the same methods of moisture retention for curing chambers and casting beds.

4.2.2.3.2 For beds cured by steam or radiant heat, cylinders shall be placed at random points along the bed. If there is any indication of variable heat, cylinders shall be placed in the coolest area.

4.2.2.3.3 Test cylinders to indicate compliance with specified 28-day or earlier strength shall remain in the bed with the member until the member is removed. At that time the cylinders shall be removed from their molds and placed in storage in a moist condition at $73.4^\circ \pm 3^\circ \text{F}$.

4.2.2.4 Capping of Cylinders. Unless cylinder ends are cast or ground to within 0.002 inch of a plane surface, they shall be capped prior to testing. Capping procedures shall be as specified in ASTM C 31 and C 617 except with fast setting sulphur compounds, especially manufactured for capping, compression testing can be performed one half hour or less after the caps have been in place. Neoprene bearing pads may be used providing the equipment and procedures are approved by the Engineer.

4.2.2.5 Testing of Cylinders.

4.2.2.5.1 Testing of cylinders to determine compressive strength shall be performed in accordance with ASTM C 39. The strength of concrete at any given age shall be determined as the average of two cylinders.

4.2.2.5.2 Testing machines shall be calibrated so the maximum error is not more than ± 1 percent. Calibration shall be performed whenever there is a reason to doubt the accuracy of indicated loads, or at least once every six months. Calibration curves shall be available at all times and used by testing personnel.

4.2.3 Air Content. The air content tests shall be conducted in accordance with ASTM C 231. At least one air content test will be conducted on the concrete from which each member is cast.

5. EVALUATION AND ACCEPTANCE.

5.1 Concrete. A test result shall be the average of the strengths of the two test cylinders made in accordance with paragraph 4.2.2.1. The strength level of the concrete will be considered satisfactory if the average of all sets of three consecutive

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strength tests equal or exceed the specified strength f_c and no individual test falls below the specified value by more than 500 psi. Members manufactured with concrete which does not meet the strength requirements shall be rejected. All members cast with concrete having a measured air content less than 5 percent shall be rejected.

5.2 Tolerances. The precast members shall be manufactured within the following tolerances. Members which fail to meet the dimensional tolerances shall be rejected.

5.2.1 Length of Member. The length of the member shall not deviate from the length shown on the contract drawings by more $\pm 1/8$ inch per 10 feet of length, maximum of $\pm 1/2$ inch.

5.2.2 Width of Members. The width of a member, shall not vary by more than $\pm 1/4$ inch.

5.2.3 Horizontal Alignment (Sweep). The horizontal alignment of the members shall not deviate from a straight line parallel to the theoretical centerline by more than $1/2$ inch or $1/8$ inch per 10 feet of length, whichever is greater.

5.2.4 Handling Devices. The Contractor will be responsible for the design and positioning of all handling devices. The actual position of handling devices shall not deviate from the designed position by more than ± 6 inches.

5.2.5 Anchors and Inserts. The actual position of anchors and inserts shall not vary by more than $\pm 1/4$ inch from positions shown on the contract drawings.

5.2.6 Slab Thickness. The thickness of a slab shall not vary from the dimensions in the drawings by more than $\pm 1/4$ inch.

5.2.7 Planeness. The planeness as measured with respect to a straight line drawn between any two opposite edges shall be $\pm 1/4$ inch for the outside surface and $\pm 1/2$ inch for the inside surface.

5.2.8 Squareness of Ends. The ends of members shall not deviate from being square by more than $\pm 1/8$ inch. Squareness shall be checked in both the vertical and horizontal planes.

5.3 Defects.

5.3.1 Minor Defects. Minor defects are those which involve less than 36 square inches of concrete, and do not expose reinforcing steel. These defects will be repaired as specified hereinafter. Cracks which are visible but are .006 inch wide or less will be accepted.

5.3.2 Major Defects. Major defects are those which involve more than 36 square inches of concrete or reinforcing steel. If one or more major defects appear in a member it shall be rejected. Cracks of a width of more than .006 inch shall be cause for rejection of the member.

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6. SUBMITTALS.

6.1 Design Calculations. Design calculations for members and connections not indicated shall be submitted to the Contracting Officer prior to the initiation of manufacture of members to be used under this contract.

6.2 Test Reports. Certified test reports of required material tests shall be submitted to the Contracting Officer prior to the use of the materials in the work. Reports shall be furnished for each shipment and shall be identified with specific lots.

6.3 Test Results. The results of construction testing by the Contractor shall be submitted to the Contracting Officer not more than 5 days after the tests are completed.

6.4 Manufacturer's Certificate.

6.4.1 Cement. Cement shall be certified for compliance with all specifications requirements, including those for microsilica content where included.

6.4.2 Air-entraining admixture shall be certified for compliance with all specification requirements.

6.4.3 Water-reducing admixture shall be certified for compliance with all specifications requirements.

6.4.4 Aggregates shall be certified for compliance with all specifications requirements.

6.4.5 Superplasticizers. If the Contractor proposes to use a high range water reducing agent (HRWRA) or superplasticizer, the Contractor at his expense shall test the superplasticizer for compressive strength (ASTM C39) and freeze-thaw durability (ASTM C666 Procedure A) utilizing concrete mix materials that are to be used on the project. If the Contractor wishes to use superplasticizer in the concrete mix before these test results can be obtained, he shall meet the additional requirements as follows. The Contractor shall test, at his expense, hardened concrete samples utilizing project materials and the intended superplasticizer for a microscopic air-void determination (ASTM C457). The concrete samples to be tested shall be made in the presence of the Contracting Officer. The superplasticizer shall be accepted as satisfactory if the spacing factor is less than 0.0078 inches, the air-void specific surface is greater than 600 sq. in. per cubic inch of air void volume, and the number of voids per linear inch of traverse shall be greater than 1.5 times the numerical value of the percentage of air in the concrete. The air content (ASTM C231) of the superplasticized concrete samples shall be determined. Superplasticized concrete to be placed in the panels shall have an air content range within $\pm 1\frac{1}{2}\%$ of the tested concrete samples.

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6.4.6 Certified Air Content. Each precast member delivered to the job site shall be accompanied by a certificate certifying that the air content in the concrete in that member is in compliance with the specifications. The certification must be based on an air content test conducted in conformance with ASTM C 231 on at least one of the batches concrete from which the member was cast.

6.5 Concrete Mixture Proportions. Concrete mixture proportions shall be submitted to the Contracting Officer for approval. A statement giving the maximum nominal coarse aggregate size, the proportions of all ingredients and the type and amount of any admixtures that will be used in the manufacture of each strength and type of concrete shall be furnished. The statement shall be accompanied by test results from an approved testing laboratory, certifying that the proportions selected will produce concrete of the properties required. No substitutions shall be made without additional tests to verify that the concrete properties are satisfactory.

6.6 Shop Drawings. The Contractor shall prepare and submit to the Contracting Officer for approval, complete shop drawings which show the precast unit manufacturer's recommended details and materials for the work required by paragraph HANDLING AND ERECTION and the following:

- a. Design Computations.
- b. Marking of the units for the placing drawings.
- c. Anchorages for lifting.
- d. Anchorages for work of other trades.
- e. Anchorages to support construction.
- f. Location and sizes of all blockouts to be cast into members.
- g. Formwork.
- h. Joints between units and other construction.
- i. Reinforcing steel details.
- j. Method of curing.
- k. Pickup points and lifting devices.
- l. Provisions for maintenance of power culvert drains (River Wall, Monoliths 44/45 and 55).

6.7 Manufacturer's Qualifications. Prior to commencing operations a statement giving the qualifications of the precast concrete manufacturer shall be submitted.

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7. PRODUCTS

7.1 Design

7.1.1 Standards and loads. Precast unit design shall conform to ACI 318. Stresses due to restrained volume change caused by shrinkage and temperature differential, handling, transportation and erection shall be accounted for in the design. Loadings for all members shall include all dead, live, handling, erection, and other applicable loads.

7.1.2 Allowable Crack Widths. Provide sufficient reinforcement in exposed panel faces such that maximum allowable crack width on exposed face of erected panels shall be less than or equal to 0.006".

7.1.3 Calculations. Calculations for design of members and connections not indicated shall be made by a registered professional engineer experienced in the design of precast concrete.

7.2 Material.

7.2.1 Portland Cement shall conform to ASTM C 150 Type II.

7.2.2 Aggregates shall be produced from the sources and under the conditions described in paragraph 3.1.1 of Section 03300, CAST-IN-PLACE STRUCTURAL CONCRETE. Fine and coarse aggregates shall conform to the gradation requirements of ASTM C-33, or NYSDOT approved gradation. Course aggregate shall have a Mohs hardness of 5 or greater.

7.2.3 Admixtures to be used, when required or permitted, shall conform to the appropriate specification listed below:

7.2.3.1 Air Entraining Admixture. ASTM C260

7.2.3.2 Accelerating Admixture. Calcium chloride or other accelerating admixtures shall not be allowed.

7.2.3.3 Water-reducing or retarding admixtures ASTM C 494, type A, B, or D.

7.2.3.4 High Range Water Reducer. ASTM C 494, type F or G.

7.2.3.5 Microsilica source shall be approved by the Contracting Officer. A single source shall be used throughout the project. Microsilica shall be a byproduct of the production of silicon.

7.2.3.6 Steel reinforcement shall be supplied in accordance with Section 03200 STEEL BARS AND ACCESSORIES FOR CONCRETE REINFORCEMENT.

7.2.3.7 Steel Welded Wire Fabric. "W" plain type (smooth) wire shall conform to ASTM A 82 and fabric shall conform to ASTM A185.

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7.2.3.8 Embedments shall be hot-dip galvanized.

7.3 Composition and Quality.

7.3.1 Concrete shall be composed of portland cement, water, fine and coarse aggregate, and admixtures. The admixtures shall be microsilica, high range water reducer, and an air entraining agent.

7.3.2 Quality. the concrete proportions shall be selected by the Contractor to meet the following requirements:

Maximum Water Cement Ratio = 0.40
Compressive Strength = 7,000 psi at 28 days
Entrained Air = 5 to 7 percent
Microsilica = 7.0 percent by weight of cement maximum (optional)

Proportions shall be selected so that the maximum permitted water-cement ratio is not exceeded and so as to produce an average strength exceeding the design strength f'_c by the amount indicated below. Where the production facility has a standard deviation record determined in accordance with ACI 214, based on 30 consecutive strength tests of similar mixture proportions to that proposed, obtained within one year of the time when concrete placing is expected, it shall be used in selecting average strength. the average strength used as the basis for selecting proportions shall exceed the specified strength f'_c by at least.

400 psi if standard deviation is less than 300 psi
550 psi if standard deviation is 300 to 400 psi
700 psi if standard deviation is 400 to 500 psi
900 psi if standard deviation is 500 to 600 psi

If the standard deviation exceeds 600 psi or if a standard deviation record is not available, proportions shall be selected to produce an average strength at least 1200 psi greater than the specified strength.

The Contractor shall have the mixture proportions determined by persons knowledgeable in the formulation of concrete mixtures. The trial mixtures shall be formulated using the same materials as those to be used in the units supplied under this specification and the selected proportions shall be submitted for approval with the results of cylinder strengths at 7 and 28 days.

8. FABRICATION

8.1 BEDS AND FORMS.

8.1.1 All casting beds shall have concrete support on unyielding foundations.

8.1.2 Forms, both fixed and movable, shall be of steel of adequate thickness, or formica faced plywood braced, stiffened, anchored and aligned adequately to produce members within the limits of dimensional tolerances specified hereafter.

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8.1.3 Bulkheads, spacers, templates and similar equipment having influence on the accuracy of dimensions and alignment shall be regularly inspected and maintained as necessary.

8.1.4 Accurate alignment of forms shall be maintained during the casting operation. Form, alignment and grade shall be checked for each setting. Form joints shall be smooth and tight enough to prevent leakage of paste. Joints between soffits, side forms and bulkheads may require gaskets of rubber or other suitable material to prevent leakage of paste. Such material may be used to provide corner chamfers. Plugging of holes and slots in the forms shall be done neatly so the finished member will have a workman like appearance.

8.1.5 For exposed members, form ties, if used, shall be of the threaded or snap-off type so no metal parts will be left at the surface of the finished concrete.

8.1.6 Provision shall be made in form anchorage for any anticipated differential movements of beds and forms during the casting and curing operations.

8.1.7 Beds and form shall be thoroughly cleaned after each use. Coatings used for release of members shall not be allowed to build up.

8.2 STEEL REINFORCEMENT. Steel bars and welded wire fabric shall be placed in accordance with Section 03200, STEEL BARS AND ACCESSORIES FOR CONCRETE REINFORCEMENT.

8.3 EMBEDDED ACCESSORIES. Anchors, inserts, lifting devices, and other accessories which are to be embedded in the precast units shall be furnished and installed in accordance with the approved shop drawings. Embedded items shall be accurately positioned in their designed location, and shall have sufficient anchorage and embedment to satisfy design requirements.

8.4 CONCRETE PLACEMENT. Concrete placement shall be in accordance with Section 03300 CAST-IN-PLACE STRUCTURAL CONCRETE, except that once placement is started in a member it shall be carried on in a continuous operation until the member is completed. Members shall be cast in a horizontal position and casting in tiers will not be permitted. Adequate vibration shall be provided with internal and form vibrators so the cast members shall be free of rock pockets or surface blemishes resulting from inadequate vibration. Cold joints shall not be permitted. If delays occur which result in hardening of the concrete so it will not receive a vibrator and again become plastic, partially filled forms shall be washed out or partially cast members rejected.

8.5 CURING AND PROTECTION. Concrete for the manufacturing of the precast concrete members shall be cured and protected in accordance with Section 03300, CAST-IN-PLACE STRUCTURAL CONCRETE or by other methods further specified herewithin. The precast members shall be moist-cured until the concrete reaches a minimum strength of 0.7f_c.

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8.5.1 Curing with Steam at Atmospheric Pressure. Steam curing shall be under a suitable enclosure to retain the live steam to minimize moisture and heat losses. The enclosure shall allow free circulation of the steam around the sides and top of the panels. Steam jets shall be so positioned so they do not discharge directly on the concrete, forms, or test cylinders. The cycle of steam application shall conform to the following:

8.5.1.1 After placing and vibrating, the concrete shall be allowed to attain its initial set before the steam is applied. During the period between placement of the concrete and application of steam, provisions shall be made to prevent surface drying by means of a coating of membrane curing compound, moist covers or equally effective methods. Application of the steam shall be delayed not less than 2 hours and nor more than 10 hours after the time of concrete placement. If the ambient temperature is below 50°F enough heat shall be applied to maintain the concrete at its placing temperature.

8.5.1.2 The ambient temperature within the casting enclosure shall be increased at a rate not to exceed 40°F per hour. Temperature increase shall be as uniform as possible.

8.5.1.3 The temperature shall be increased until the ambient temperature in the casting enclosure is between 140°F and 160°F.

8.5.1.4 In discontinuing the steam curing the ambient air temperature shall decrease at a rate not to exceed 40°F per hour. Temperature decreases shall be as uniform as possible.

8.5.1.5 If Steam is Used, Recording thermometers showing the time-temperature relationship through the curing period shall be provided. At least one recording thermometer per casting enclosure shall be used. The desired curing time-temperature relationship shall be placed on the recording chart of the recording thermometer, to aid the personnel that control the temperature during curing. Recording charts shall be made available upon request and shall be clearly visible during the curing process.

8.5.2 Curing with Radiant Heat and Moisture.

8.5.2.1 Radiant heat may be applied to beds by means of pipe circulating steam, hot oil or hot water or by electric blankets or heating elements on forms. Pipes, blankets or elements shall not be in contact with concrete, form surface or test cylinders.

8.5.2.2 During the cycle of radiant heat curing, effective means shall be provided to prevent rapid loss of moisture in any part of the member. Moisture may be applied by a covering of moist burlap or cotton matting. Moisture may be retained by covering the member with a plastic sheet in combination with an insulating cover, or by applying a liquid seal coat or membrane curing compound.

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8.5.2.3 Temperature limits and use of recording thermometer shall be as specified for curing with steam at atmospheric pressure.

8.5.2.4 Termination of curing shall be as specified in Section 03300, CAST-IN-PLACE STRUCTURAL CONCRETE unless the concrete has been cured by one of the two methods stated above. Termination of curing for concrete cured by either the steam at atmospheric pressure method or the radiant heat with moisture shall be determined based on the compressive strength of the concrete necessary for stressing or destressing the tendons. The strength shall be determined from concrete cylinders as specified in paragraph entitled QUALITY ASSURANCE.

8.6 REPAIRS. All honeycombed areas, chipped corners, air pockets over 1/4 inch in diameter, and other minor defects shall be repaired. Form offsets of fins over 1/8 inch shall be ground smooth. All unsound concrete shall be removed from defective areas prior to repairing. All surfaces permanently exposed to view shall be repaired by a blend of portland cement and white cement properly proportioned so that the final color when cured will be the same as adjacent concrete.

8.7 FINISHING. Precast panels shall have a smooth dense finish on the outside, exposed surface such as is typical of steel form or high density overlaid plywood forms. The inside panel surface shall be clean, free of laitance, and shall be intentionally roughened to an approximate amplitude of 1/4 in. The surface shall be cleaned by high-pressure water spray immediately prior to erection.

9. HANDLING AND ERECTION.

9.1 GENERAL. The manufacturer shall demonstrate and submit for approval the strength required to remove the panels from the form based on allowable crack widths of 0.01" for lifting/handling operations. The location of pickup points for handling of the members and details of the pickup devices shall be shown on shop drawings. Members shall be handled only by means of approved devices at designated location.

9.2 POSITION OF MEMBERS. In the handling of members, it is imperative they be maintained in an upright position at all times and picked up and supported as shown on approved shop drawings.

9.3 STORAGE.

9.3.1. Storage areas for precast members shall be stabilized, and suitable foundations shall be provided, so differential settlement or twisting of members will not occur.

9.3.2 Stacked members shall be separated and supported by battens placed across the full width of each bearing point. Battens shall be arranged in vertical planes at a distance not greater than the depth of the member from designated pickup

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points. Stacking of members shall be such that lifting devices will be accessible and undamaged. The upper members of a stacked tier shall not be used as storage areas for shorter members or equipment.

9.4 TRANSPORTATION.

9.4.1 In transporting members by truck, railroad car or barge, provision shall be made for supporting the members as described above, except battens can be continuous over more than one stack of units, with adequate bracing to ensure their maintaining the vertical position and damping of dangerous vibrations. Trucks with double bolsters are generally satisfactory provided the members are fully seated on the outer bolsters at not more than 3 ft. or the depth of the member from the end and the inner bolster is not more than 8 ft. from the end of the member or the designated pickup point. Adequate padding material shall be provided between tie chains or cables to preclude chipping of concrete.

9.4.2 Any noticeable indication of lateral deflection or vibration during transportation shall be corrected by rigid bracing between members or by means of lateral trussing.

9.5 ERECTION.

9.5.1 All provision for storage and handling given in the foregoing paragraph shall be observed at the erection site.

9.5.2 The precast concrete members shall be set into place in a manner which assures full bearing. If the bearing called for on the contract drawing is not obtained then the members shall be removed and the situation corrected. Location Tolerances for precast panel erection shall be as follows:

Plumbness or vertical alignment: $\pm 1/2$ in.
Variation in horizontal alignment: $\pm 3/8$ in.
Precast element joint to joint alignment
Horizontal joints: $\pm 1/8$ in.
Vertical joints: $\pm 1/8$ in.

9.5.3 The Contractor shall prepare a detailed erection plan which will be submitted to the Contracting Officer at least 60 days prior to the date that erection of members is to begin. This plan shall be in sufficient detail so that adequacy of equipment, techniques and accessories can be determined and comments offered. Acceptance of the Contractor's erection plan shall not relieve the Contractor of his responsibility for erecting precast prestressed members into position as required by the plans and specifications.

10. QUALITY CONTROL.

10.1 GENERAL. The Contractor shall establish and maintain quality control to assure compliance with Contract requirements and shall maintain records of his quality control for all construction operations required under this section.

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10.2 RECORDS. Complete records shall be kept of the manufacturing, handling and erection of the precast concrete members. Records shall be kept for but not limited to the following items:

10.2.1 Specifications of material used in the manufacture of the members.

10.2.2 Records of the inspection of the members each time they are moved.

10.2.3 Records of any defects in the member and any corrective measures taken.

11. PAYMENT. Payment for the work covered by this section will be made at the contract unit price per square foot for the items listed below. Such prices shall include full compensation for all plant, labor, materials and supplies required to complete the work as shown on the plans and in accordance with the specifications.

Item 03401-1 Precast Panels, River Walls

Item 03401-2 Precast Panels, Lock Chamber Walls

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REPORT DOCUMENTATION PAGE

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13. ABSTRACT (Maximum 200 words) <p>Prior to 1988, the general approach to lock wall rehabilitation was to remove 1 to 3 ft of concrete from the face of the lock wall and replace it with new air-entrained concrete. One of the most persistent problems using this approach was cracking in the replacement concrete. Since that time, a new rehabilitation method using precast concrete as a stay-in-place form for the replacement concrete has been developed. This rehabilitation method was used initially at Lock 22 on the Mississippi River and more recently at Troy Lock on the Hudson River.</p> <p>This report is a product of the after-action study performed on the use of precast concrete stay-in-place forming system for lock chamber rehabilitation at Troy Lock and comparison with previous cast-in-place repairs. Listed herein is pertinent background information, design criteria, panel details, production data, installation details, and quality and economic comparisons with cast-in-place rehabilitation. A separate documentary video of precast concrete panel product and installation was also produced as part of the after-action study.</p> <p>The precast system used at Troy Lock was installed at a cost slightly lower than that of cast-in-place systems used previously, but with substantial improvements in appearance, durability, speed of construction, and extent of cracking.</p>				
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